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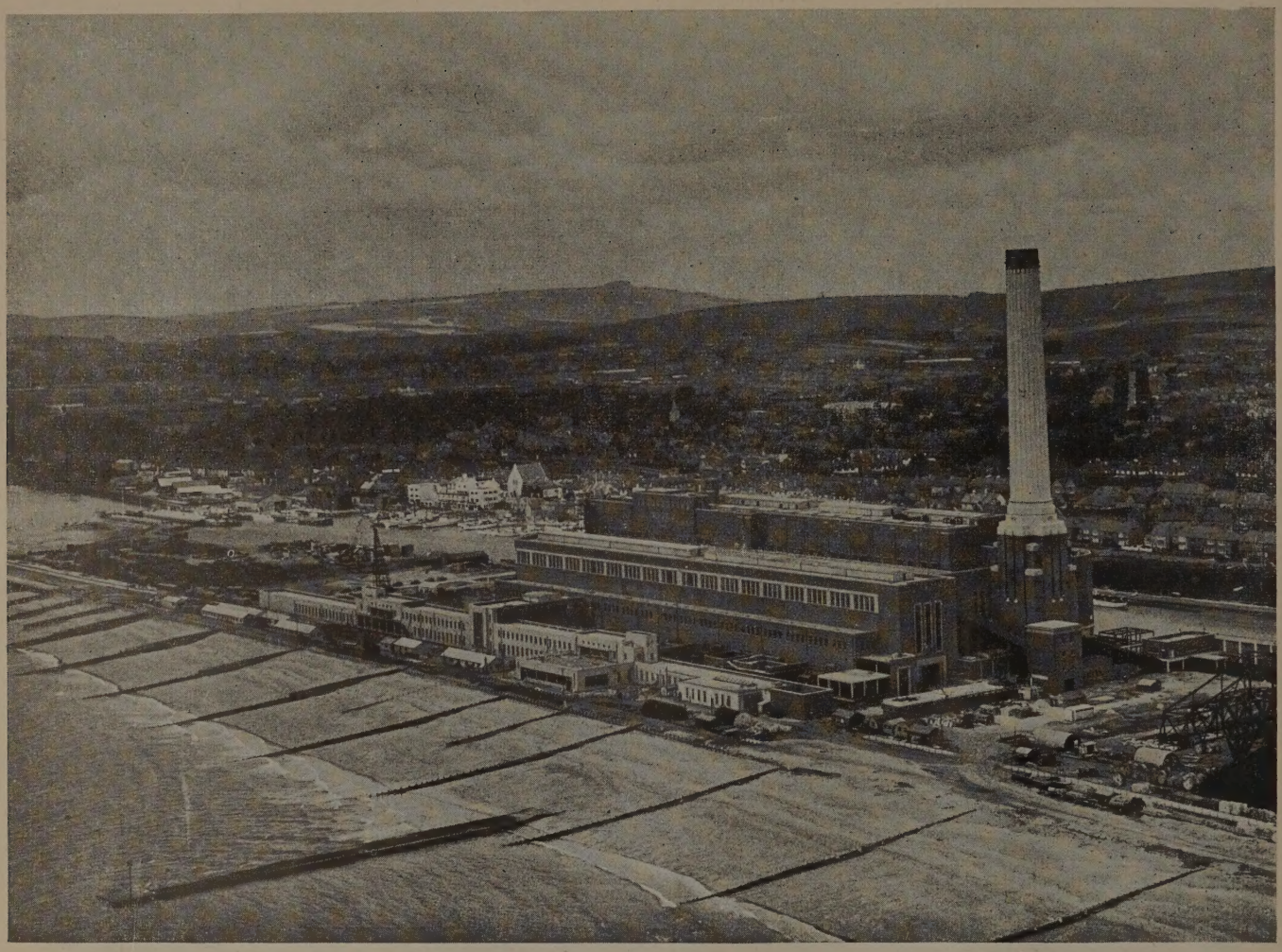

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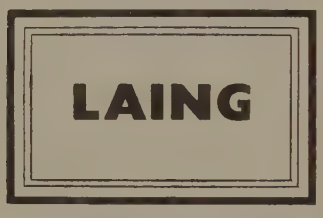

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The Construction of Eight Prestressed Concrete Tanks*

By Colonel A. Raymond Mais, O.B.E., T.D. and A. C. Little

General Requirements

In 1950 the Ministry of Works called for competitive designs and estimates for eight prestressed concrete tanks, each tank to have a capacity of 430,000 gallons, with a maximum depth of liquid of 33 feet, giving an internal diameter of 50 ft. 0 in.

Owing to the nature of the liquid to be stored in the tanks, it was a fundamental requirement that they should be impervious to liquid of a specific gravity of 1.03, under all conditions of loading and weather. For this reason, it was specified that the walls must be designed and constructed so as to be monolithic with the floors, and the usual form of sliding joint was not permitted.

The decision to use prestressed concrete in one form or another was due to the fact that it was considered to be a suitable form of construction to meet the requirement of watertightness.

Although the original specification did not require the concrete floor to be prestressed, this was later incorporated in order to guard against cracks at the joints due to shrinkage, temperature or differential settlement of the base.

Design

With " fixity " between the walls and floors, as with traditional reinforced concrete tanks, a large part of the load in the bottom section of the tank wall is carried by cantilever action instead of by direct hoop tension. This increases the bending moments carried by the wall, hence increasing the number of vertical prestressing cables, and reduces the hoop tension near the base, thus reducing the number of horizontal prestressing cables at this point.

The difficulty from the design point of view with a " fixed " joint at the base is to decide exactly how much restraint the base slab offers to the wall. A cylindrical wall is a very stiff member compared with a circular flat slab, therefore the condition of full restraint may not be achieved. If a condition of full fixity at the base of the wall were adopted in the design too many vertical prestressing wires would be required. In this design sufficient vertical prestress was provided to allow for the estimated degree of restraint at the base, and additional reinforcement was provided in the form of mild steel stirrups projecting from the floor slab into the wall, to take the difference between the fully fixed moment and the moment provided for by the vertical prestressing cables. Another governing factor for the amount of vertical prestress is the bending moment, of opposite sign to the " fixed " moment, which occurs higher up the wall. This moment, from the horizontal prestressing wires, tends to produce tension on the inside face and must be taken care of by vertical prestressing.

Another effect of the " fixed " joint between the wall and the floor is the transfer of prestress from one to the other. If the floor cables are stressed first then, due to the elastic shortening of the floor slab, the lower few feet of the wall are also partially prestressed. This loss of prestress is practically negligible if contraction gaps in the wall are left open until after the floor is prestressed. But, since the floor is prestressed before the wall, contraction gaps cannot be left open in the floor, and therefore, while horizontally prestressing the wall, there is bound to be a loss of prestress from the wall into the floor. The radial shortening due to horizontal stressing of the wall is greater than that due to stressing the floor and, in these tanks, the loss of prestress from the walls into the floor was compensated for by adding extra horizontal cables near the base of the wall.

It should be noted that in a prestressed, or reinforced concrete, tank with a " fixed " joint at the base, cracking is most likely to occur horizontally near the bottom of the wall—in theory, along the junction of the wall and base. But in practice, partly because of the upstand and partly because of the neglected tensile strength of concrete, it is most likely to occur along the lowest horizontal construction joint in the wall where the tensile strength of the concrete is very much reduced, particularly if this joint is within 1 ft. 0 in. of the floor.

The advantage of a prestressed tank is that the tendency to crack is greatest before the tank is filled. The moment at the base caused by the horizontal prestressing cables is greater than the moment, of opposite sign, due to the liquid loads (under full liquid load there is still 150 lb. per sq. in. residual prestress). This moment from the horizontal prestressing cables tends to produce tension on the outside and compression on the inside face near the base of the wall and the liquid loads only reduce the tension and compression but do not reverse them. Thus, the liquid face is always under compression. The prestressed tank thus undergoes its severest test before the liquid load is ever applied.

Concrete Stresses

(lb./sq. in.)

The following stresses assume that all the usual losses of prestress have taken place :—

	Tank Empty	Tank Full
Hoop compression in wall	630 (near base) 225 (near top)	150 (minimum residual near base) 225 (do. near top)
Vertical compression in wall	425 max. 50 min.	425 max. 290 min.
Compression in base slab	300 max. 30 min.	300 max. 200 min.

The figures given for the residual prestress when the tank is full, in the wall (vertical), and the base slab, depend entirely on the degree of fixity provided at the junction. This is indeterminate and the figures given

Paper to be read before a Joint Meeting of the Institution of Structural Engineers and the Reinforced Concrete Association at 11, Upper Belgrave Street, London, S.W.1., on Thursday, 8th January, 1953, at 6 p.m.

for residual prestress are based on the estimated degree of restraint used throughout the calculations.

Theoretically, if Young's Modulus for the concrete is 4×10^6 lb. per sq. in. :—

Due to vertical prestress, shortening of wall height

$$= \frac{425}{4 \times 10^6} \times \frac{420}{1} = 0.045 \text{ in.}$$

Due to prestress in floor, radial contraction of base slab

$$= \frac{300}{4 \times 10^6} \times \frac{336}{1} = 0.025 \text{ in.}$$

Due to horizontal prestress, radial contraction

$$= \frac{630}{4 \times 10^6} \times \frac{309}{1} = 0.049 \text{ in.}$$

These figures will tend to increase due to creep and shrinkage under the prestressing load.

Wire Stresses

It may be seen from the above that the strains involved in the concrete are very small and hence the wire stresses will vary very little between the " tank empty " and " tank full " conditions—actually less than 3 per cent. Consequently, the wire stresses which matter are the anchoring stresses and the residual stresses with the tank empty after all losses have taken place. With the tank full, the wire stresses will be up to 3 per cent. greater than these residual stresses.

	Anchoring Stress. lb/ins. sq.	Residual Stress. lb./ins sq.
Vertical cables	130,000	114,000
Floor cables	130,000	114,000
Horizontal cables (2 pt. stressing)	130,000 (at sandwich plate)	
	100,000 (mean stress)	87,000 (mean stress)
Horizontal cables (1 pt. stressing)	79,000 (mean stress)	68,000 (mean stress)

General Description of Tanks

The form of construction decided upon was as follows. The tank base was of prestressed concrete 10 in. thick, increased to 2 ft. 0 in. round the periphery. In plan it was roughly in the form of an octagon on axes 56 ft 0 in. long (see Fig. 2). The prestressing cables were arranged to run right through the tank base in two directions at right angles, those in one direction running through the middle of the 10 in. slab, and the remainder at right angles running alternately above and below the cables in the first layer. The cables in the base each consisted of eight wires 0.276 in. diameter, and were spaced at approximately 1 ft. 5 in. centres in each direction, the layout being completely symmetrical (see Figs. 2 and 5).

The tank walls were 9 in. thick, and 34 ft. 7 in. high, with a ring beam 1 ft. 3 in. deep and 1 ft. 6 in. wide at the top to carry the thrust from the domed roof. Prestressing was imparted by means of vertical cables, each of eight wires 0.276 in. diameter passing through the centre of the walls at 1 ft. 3 in. intervals and anchored in the tank base, together with circumferential cables, again with eight wires each, passing round the outside of the tank at vertical spacing varying from 15 in. at the top of the walls to 6 in. at the base.

The domed roof was constructed of R.S.J.s, curved in elevation and cased in concrete, carrying precast concrete units, over which a concrete screed was laid. (see Figs. 1 and 4).

Planning I

In the planning stage, some anxiety was felt over the possibility of leaks through cracks caused by shrinkage contraction of the concrete, both in the walls and the base.

Theoretically the prestressing should close these cracks, although at the same time this would reduce the residual stress in the concrete, but it was felt that any cracking was undesirable in these tanks even if it were subsequently eliminated by stresses. It was finally decided to adopt the following measures. The base was divided into four quadrants, each concreted in as short a time as possible, a period of at least three clear days being allowed to elapse between placing concrete in adjacent quadrants.

The walls were concreted in 3 ft. 3 in. lifts, and four gaps 9 in. wide were left at points 90 degrees apart, for the entire height of the wall, and straddling the joints between the quadrants in the base. These gaps were filled later, each lift of the construction gap being filled not less than ten days after the corresponding lift of the main wall.

These gaps introduced many constructional difficulties and, as they subsequently proved to be the only real weaknesses in the tanks, they were eventually omitted, and in the last three tanks each lift was carried out as a continuous process, and proved entirely satisfactory in every respect.

Owing to the dry concrete necessary to meet requirements of strength, considerable thought was given to satisfactory methods of compaction. A 20 ft. 0 in. square " mock-up " of the base slab was constructed, complete with rubber cores and B.R.C. fabric, and was concreted using various methods of vibration—*e.g.*, immersion vibrators, of different types, pan vibrators, and a vibrating screed. It was found that the most efficient compaction was obtained by using an electric immersion vibrator with a short 3 in. diameter head, and finishing off with a vibrating screed. The latter was made up on the site, consisting of a 16 ft. 0 in. length of 5 in. × 3 in. R.S.J., with raised " bicycle " type handlebars at each end, and carrying two Sinex vibrators. This imparted a very satisfactory finish to the concrete surface, in addition to compacting the top few inches of the slab.

On the question of compaction of concrete in the walls, it was found that two schools of thought existed, one favouring the use of Sinex vibrators clamped to the shutters, the other favouring immersion vibrators. It was finally decided to use the latter method, partly owing to the large number of Sinex vibrators required and the high initial cost—whereas the number of immersion vibrators required was small by comparison, and available from stock—and partly because it was felt that there was a possibility of shutters slipping after being fixed if external vibrators were mounted on them, and that the lift of the shutters would be reduced. It was realised that the use of immersion vibrators would require very careful supervision of the labour handling the tools to ensure adequate compaction, and this was accepted. As a matter of interest, it was found on completion of the job that in only one small section of the wall, approximately 9 in. square, was there any evidence of insufficient compaction of the concrete, and on further investigation, it was found that this section of the work had been carried out by a " scratch " concrete gang working during a holiday period.

Owing to the requirement that the tank walls should be monolithic with the base, the lower ends of the vertical

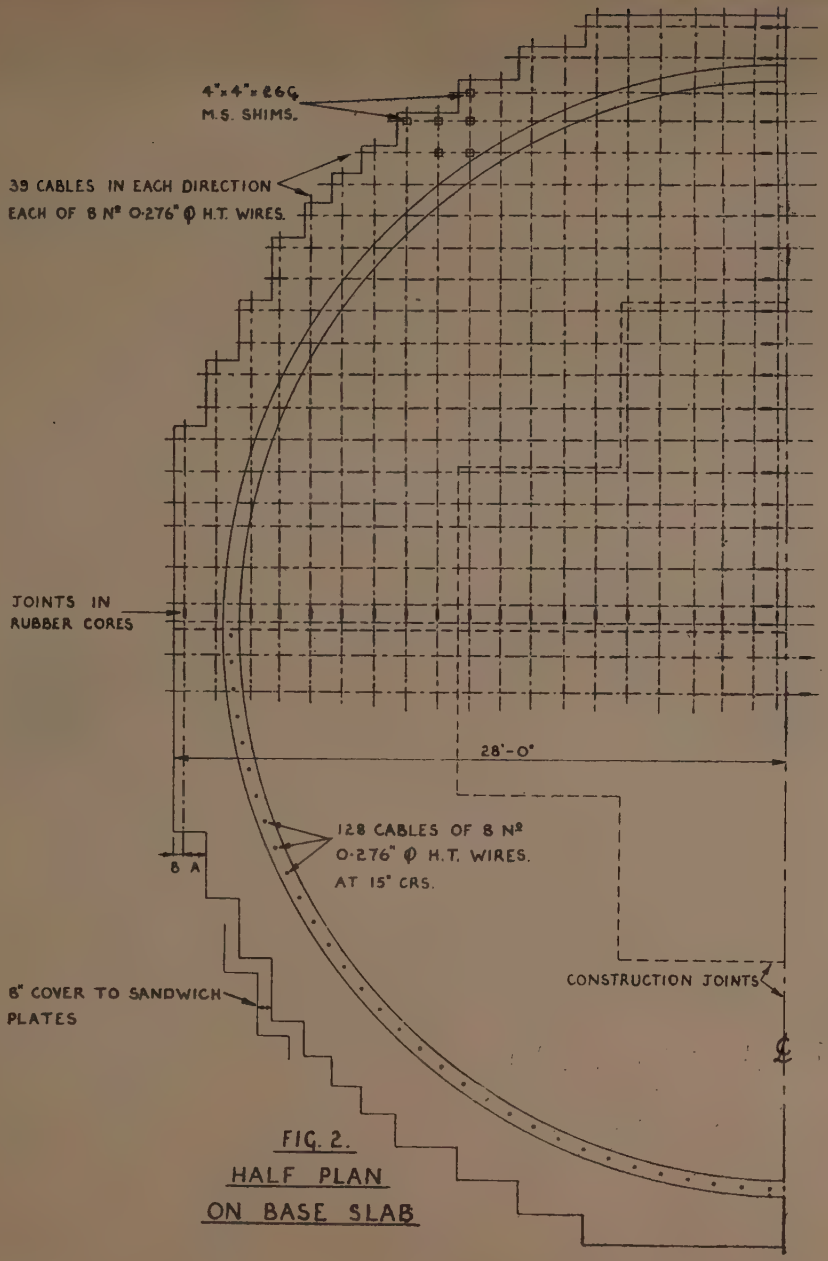
4"x4"x266 M.S. SHIMS.
39 CABLES IN EACH DIRECTION EACH OF 8 Nº 0·276" Ø H.T. WIRES.
JOINTS IN RUBBER CORES
28'-0"
128 CABLES OF 8 Nº 0·276" Ø H.T. WIRES. AT 15" CRS.
8 A
6" COVER TO SANDWICH PLATES
CONSTRUCTION JOINTS
℄

FIG. 2.
HALF PLAN
ON BASE SLAB

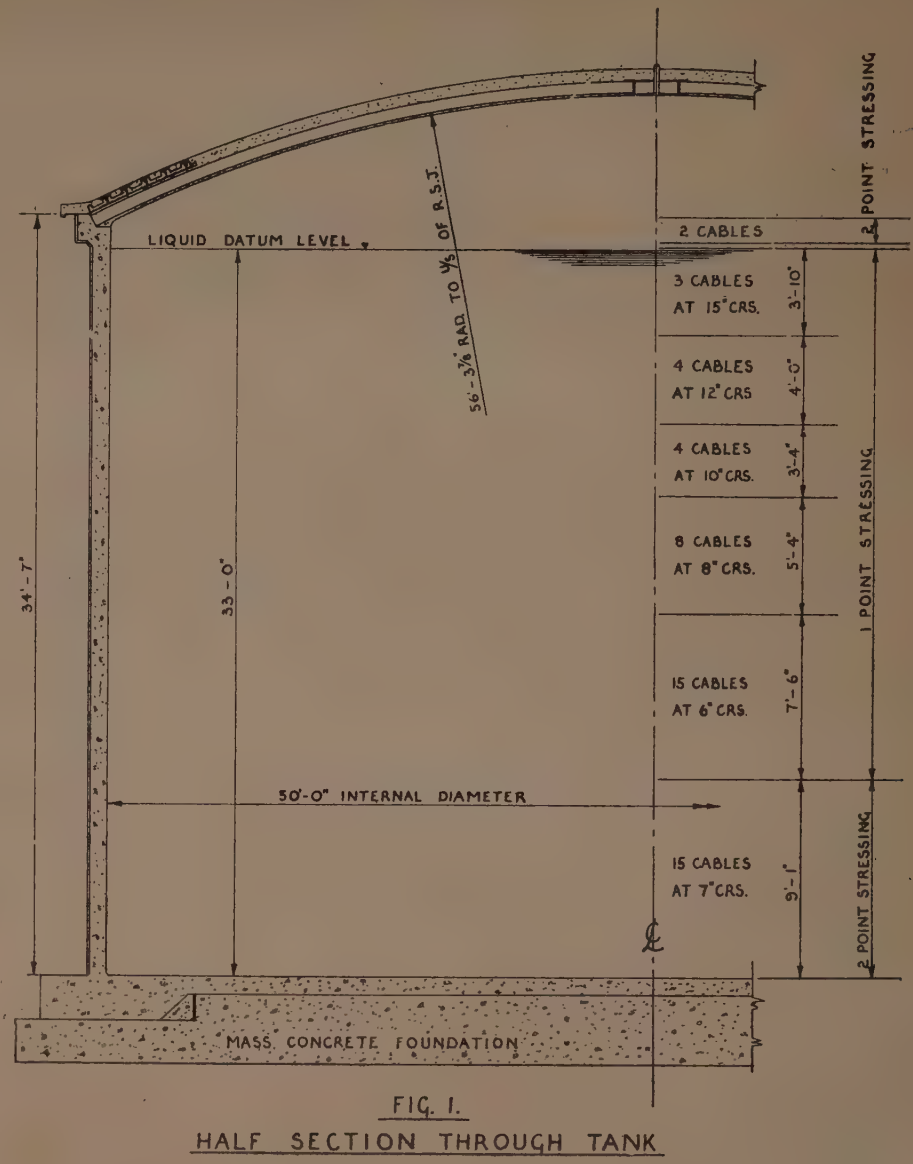

FIG. 1.
HALF SECTION THROUGH TANK

cables had to be anchored in the base slab, and a duct formed round the cable, which was already in position, whilst the tank base and walls were being concreted. It was decided to form this duct by means of a length of 2 in. O.D. steel scaffold tube, which could be well greased before concreting, and then lifted whilst the concrete was still green. It was necessary that the end of this tube should always be well down into the previous lift, in order to stop grout running down the cable duct and building up at the bottom. The tubes—128 per tank—were, therefore, made 6 ft. 0 in. long, to allow for 1 ft. 0 in. in the previous lift, 3 ft. 3 in. in the current lift, and 1 ft. 9 in. protruding from the top; this latter was more than was actually necessary for gripping and lifting the tubes, but was provided to ensure that the mouth of the tube was well clear of the concreting operation. In practice, keeping these cable ducts clear, so that subsequent grouting could be carried out successfully from the bottom, was a very real problem, necessitating constant attention.

The holes for the cables in the tank base were to be formed by the normal Magnel-Blaton extractable rubber core, in this case of 2 in. square section. This was stiffened by means of a 1 in. diameter M.S. rod passed through the central hole, after being well greased with Shell F.21 grease.

It was necessary that the perimeter of the base should be formed in a series of steps in plan, in order to form a flat surface normal to the C.L. of the cable, against which the sandwich plate could be placed. Some care was necessary over planning these steps, as the dimension A (see Fig. 2) could not be less than 8 in. in order that there would be sufficient room for the jack, and the dimension B could not be less than 5 in. to ensure that the outside leg of the sandwich plate had adequate bearing.

Concrete—Mix and Placing

The mix specified was a nominal 1:1½:3 by weight and it was originally required that proportioning should be by weight. However, a series of tests carried out by the site laboratory established that the bulking of the sand in use did not vary appreciably from a value of 23 per cent. between values of moisture contents ranging from 2 per cent. to 10 per cent. by weight and it was, therefore, agreed to use volume batching. The stone used was Northumberland Whinstone and the sand was of local origin. The average wet density of the concrete obtained was about 161 lb. per cu. ft.

The water-cement ratio of the concrete was specified as 0.45, due allowance being made for moisture in the aggregates, which were tested every morning before starting work, and again during the day if weather conditions changed appreciably.

A 10/7 Liner Cumflow mixer was used. In this respect, a series of tests had been carried out on both a normal type drum mixer, and one specially adapted for the purpose, but they were found to be inferior to the open pan type for handling the dry concrete. The calculated quantity of water was weighed out on a spring balance and added directly to the dry aggregates in the pan after they had been thoroughly mixed. For this purpose, a scaffold platform was erected beside the mixer, and from this platform, the cement also was discharged directly from the bag into the pan of the mixer.

The care taken in producing this concrete was amply rewarded by the resulting cube tests. It was specified that the compressive strength of the concrete should be not less than 5,000 lb. per sq. in. after 28 days, and in practice it was found that this strength was almost invariably exceeded after seven days, with a 28-day strength of between 7,000 and 8,000 lb. per sq. in. These test cubes were hand tamped, and cured on the site in the open, under a single layer of hessian. Using the same materials with a water cement ratio of 0.42, and with cubes made on a vibrating table, but also cured in the open, a cube strength of 10,000 lb. per sq, in. was consistently obtained at 18 days, with 28-day results reaching a maximum of 11,700 lb.

It is worthy of note that concrete of this consistency is not easy to handle; it is very "sticky," being too wet to roll, and too dry to flow, down a chute or out of a hopper or skip. Conversely, it can be piled very high in a barrow!

Preparations of Foundations

The design required foundations capable of bearing 2 tons per sq. ft. The general oversite excavations showed several large patches of bad ground, which were removed and back filled with mass concrete, and it was finally decided to form a mass concrete foundation slab over the whole site. On this slab were formed the circular upstands to fill up underneath the 10 in. tank base slab, inside the toe. Provision was made for the small, but definite, contraction of the tank base on stressing, by floating off the area of the foundation slab under the tank base, and then coating it with bituminous emulsion. In addition, an expansion joint was formed in the upstand with Flexcell boarding, thus leaving the annular prism surrounding the upstand free to give just a little as shown in Fig. 5.

Construction of the Base

(a) *Vertical Cables.* The requirement that the tank walls should be monolithic with the base meant that the 128 vertical wall cables, each of eight wires, had to be in position before any concrete could be placed in the base. The first operation was, therefore, to erect a scaffold some 40 ft. 0 in. high round the perimeter of the tank, its standards carefully spaced to avoid the steps of the base, and also to be clear of the lines of the base cables in both directions, in order that they should not foul the jacks whilst the base was being stressed. This was planned on a large scale drawing, and the positions of the standards painted on the foundation slab. Cantilever brackets were spurred out from the top of the scaffold to carry a circular timber profile running right round the job, from which were suspended the vertical cables, to pass through the centre of the wall.

The position of each of these vertical cables had also been painted on the foundation slab, as they had to be accurately located to clear the 16 points of support of the roof girders. This was subsequently simplified by making the sections of the timber profile equal in length to the distance between the roof points, and cutting the housings for the cables at their correct spacing between these points. By this method, it was only necessary to mark out the roof points on the base, and plumb up from these points to fix the profile sections.

(b) *Rubber Cores.* It was originally intended to set up the rubber cores in two opposite quadrants of the base at once, and then to withdraw them into the other two quadrants. This, however, proved to be very difficult in practice, owing to the necessity of guiding the towing rope, the core being extracted, through the wall-starter links, spaced at 4½ in. centres, and to the fact that, as it was not possible to extract the rubber core complete with its steel stiffener, the steel rod had to be withdrawn, and then later replaced when the core was in its new position. This scheme was, therefore, abandoned after the first base had been constructed, and in the remaining tanks the whole of the rubber core was set in position before any

oncrete was placed. As the lengths of rubber core btainable were not sufficient to stretch right through he base, except at the sides, it was necessary to join two engths together, which was done by means of a sleeve or gaine " of 22 g. metal, 18 in. long, fitting tightly over he junction of the two sections of core, with the steel rod of one core protruding about 1 ft. 0 in. into the end of the other, to stiffen the joint. The layout was so arranged that these sleeves lay completely within the two opposite quadrants which were to be concreted first, in order that immediately the concrete had hardened the cores could be withdrawn from those quadrants, leaving the sleeves,

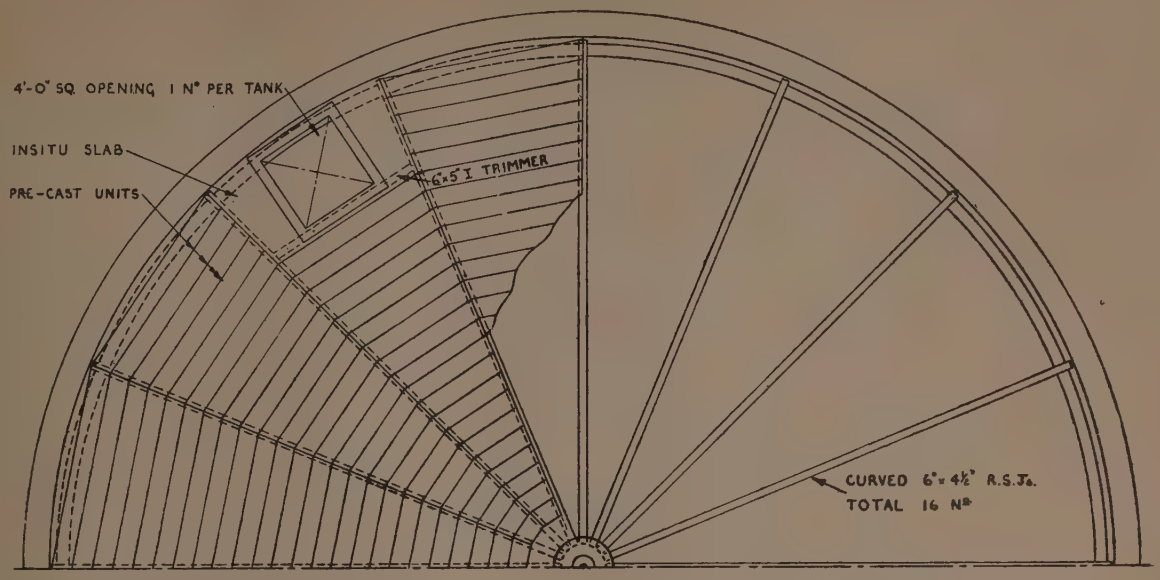

FIG. 3
HALF PLAN ON ROOF

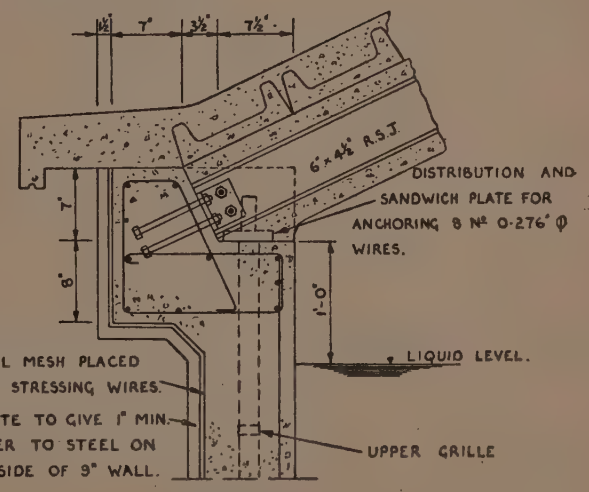

FIG. 4.
SECTION AT TOP OF WALL

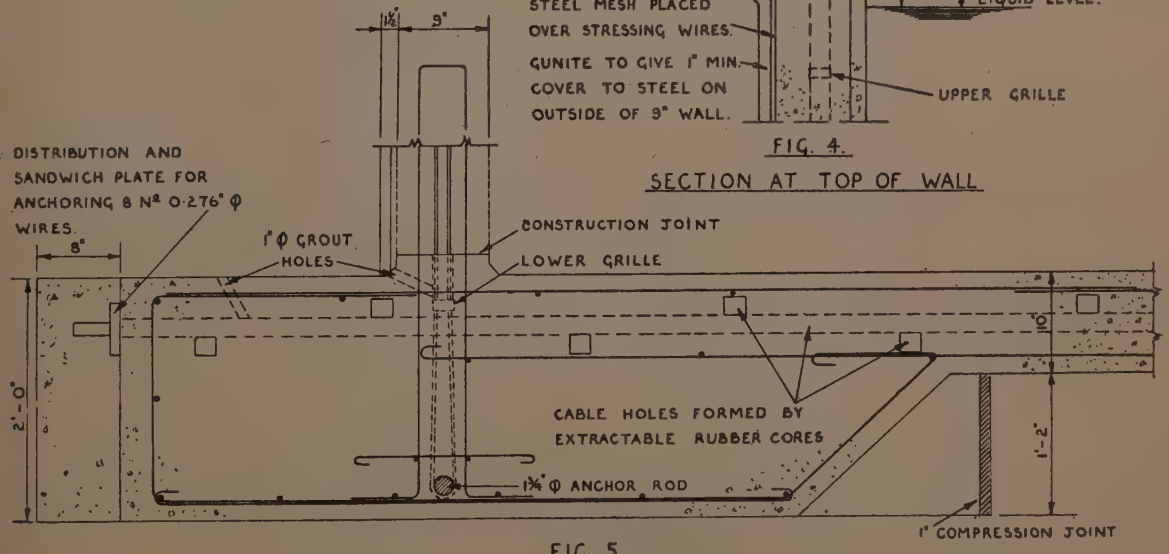

FIG. 5.
SECTION AT BASE OF WALL

with the end of the cores from the two remaining quadrants protruding into them, inside the concrete, and thus forming a grout-tight joint (see Fig. 2).

Thin metal shims 4 in. square were placed between the rubber cores at every junction. This was to prevent the grout from one cable duct running into others at the junctions, and causing a blockage. Immediately prior to concreting, the cores were well greased with Shell F.21. grease, in order to facilitate withdrawal.

(c) *Grouting Holes.* The grout holes leading to the cable ducts were formed by 1 in. diameter lengths of MS. rod, with one end cut at 45 degrees. Those leading to the floor cables were supported by timber brackets carried on the external shutters, and those feeding the vertical cable ducts were fixed in holes drilled through the external upstand shutter. Where they joined the cable duct, they were housed into a sleeve made up on site from 22 g. metal. The sleeve was also found to be a satisfactory method of forming the junction between the circular section of the scaffold tube and the rectangular section of the bottom cable grille.

The chamfered end of the rod was pushed hard against the wire, and the hole through which it was passed into the sleeve was sealed with Denso tape. It was, of course, necessary that this sleeve should be grout-tight, and the grille itself was well buttered up with a cement-sand mortar to ensure that no grout should well up through it and thus block the grout-hole. The 1 in. rods were well greased, and withdrawn immediately the initial set had taken place.

(d) *Concreting Bays.* It was not thought advisable to try to concrete a base quadrant in one operation, using stop ends to be removed as the work proceeded, and it was finally decided to divide the operation into a two-day task, each operation comprising an external section together with the wall upstand, and an internal section (see Fig. 2).

These two sections were to be concreted with the shortest possible time interval, the construction joint being well roughened, grouted, etc.

Any form of radial stop-end shutter was dismissed as being too wasteful in labour and timber, and all such shutters were made to cross the rubber cores at right angles.

(e) *Sequence of work.* While the external scaffold was being erected, the external base shutter and the majority of the reinforcement to the base, including the starter bars to the wall, were fixed. The vertical cables were then hung in position, the base reinforcement being adjusted slightly where it prevented a cable from hanging in its correct position. It was found better to do this than to hang the cables first, and then to try to fix the reinforcement round them. The rubber cores were then set in position and jointed by means of the metal sleeves, the remainder of the M.S. reinforcement then being fixed. The internal stop-end, and the shutters to the wall upstand were then fixed, together with the top mat of B.R.C. The final tasks were to fix the grout tubes to the horizontal and vertical cables, to place the shims between rubber cores at the junctions, and then, just before concreting, to coat the rubber core with Shell F.21. grease.

Barrow runs were laid on the 9 in. × 3 in. bearers which carried the upstand shutters, and the concrete was tipped straight from the barrows into the base, consolidated by immersion vibrators, and finished off with the vibrating screed as described previously.

Using three Allam electric immersion vibrators, the maximum satisfactory rate of placing the concrete was 3½ y.c. per hour. When concreting was finished, the slab was covered with hessian, which was kept wet, to assist curing, and then as soon as possible after concreting, the stop-end shutters were removed and the face of the green concrete well hacked, scarified, and cleaned off.

The rubber cores forming the cable ducts in the floors were then removed by the night shift at a minimum period of four hours after concreting had been completed. Various methods of winching were tried for removing these cores, but it was found most expeditious to attach the ends, gripped in a Post Office cable stocking, to a small-wheeled tractor, which then moved away slowly and steadily, withdrawing the rubber core in the process. Little or no damage resulted to the core as a result of this somewhat rough and ready method.

Construction of Walls

Purpose-made timber shutters were used, consisting of 1¼ in. boards mounted on walings of 6 in. × 3 in. section cut to the necessary radius. The panels were 3 ft. 9 in. deep, 4 ft. 4 in. long, and faced with 24 g. sheet metal. Forty uses were obtained from these panels which were then resheeted and used another 40 times on other work.

It was quickly found advantageous to bolt these shutters together in pairs. This facilitated setting, gave greater rigidity when erected, and reduced the time of striking and refixing. Clamps were used for fixing the shutters in position and proved most satisfactory.

Prior to concreting a lift of the walls, the surface of the previous lift was thoroughly cleaned and wetted, and covered to a depth of about 1 in. with a dry 1:1½ cement-sand mortar, which was well rammed home. The concrete was then placed in layers of about a foot, and continuously vibrated by means of Allam electric vibrators; the average rate of placing was approximately 3 y.c. per hour, *i.e.* about 4½ hours per lift.

It was found that to cope successfully with the problem of freeing and lifting the scaffold tube sleeves forming the vertical ducts through the walls, a definite technique was required, and a special gang was formed for this task alone.

As soon as concreting was finished one man moved round the walls scraping away the top 1 in. of the still green concrete. He was followed by another man with a high pressure water jet, who washed the surface of the construction joint. This washed away the fines, leaving an admirably ragged surface in which each piece of stone had its lower part firmly embedded in the parent concrete, and its upper part washed clean and protruding from the surface, ready to receive the mortar preparatory to the placing of the next lift.

While this was going on, the remainder of the gang were moving round the perimeter, twisting the tubes to break the bond, and then lifting each one about 6 in. to free it completely. This operation had to be carried out between fairly rigid time limits, as all of the 128 tubes had to be treated thus during a period of approximately one hour, after the concrete had gone off sufficiently to maintain the true shape of the duct when the tube was lifted, and before it had set so firmly as to grip the tube tightly, and prevent its easy withdrawal. It was found that weather conditions had a very great effect on these time limits. When all the tubes had been freed thus, they were lifted, scraped clean, greased, and reset for the next lift.

The vertical joints at the contraction gaps were treated in the same way as the horizontal construction joints; the shutters were removed as soon as possible after placing the lift, and the exposed face of green concrete was scarified and treated with the water jet. The contraction gaps were then filled in lifts, each lift being

aced not less than ten days after the corresponding lift the main wall.

Stressing

All stressing was carried out on the Magnel-Blaton ystem, using 0.276 in. high tensile steel wire, of 100-110 ons per square inch ultimate strength, and 70-80 tons er square inch 0.1 per cent. proof stress. This was eceived direct from the manufacturers in coils of 8 ft. 0 in. iameter, weighing about 3 cwt. each, and the stressing ables were made up on site.

(a) *Horizontal Cables.* The base cables were straightorward eight wire cables, with spacing grilles at approxiately 6 ft. 0 in. centres and varying in length to suit the articular section of the base which they were to occupy. t was found that a cable could easily be pushed into osition from one end of its duct, without the necessity f a lead wire or any form of special nose.

Normal Magnel-Blaton jacks were used, and the stressng was perfectly straightforward, with very little slip ccurring on the wires as the load was released.

The extension of the wires on stressing varied from $7/16$ in. on the shortest cables, 25 ft. 0 in. long, to $3\,7/16$ in. on he largest cables, which were 59 ft. 0 in. long.

(b) *Vertical Cables.* These were made up from four ong wires, each with a 180-degree bend at its mid point, where passing round the $1\frac{3}{4}$ in. anchor bar.

The end of each wire was then passed successively hrough the lower grille, the tube used to form the cable luct, and the upper grille (Figs. 4 and 5), forming in ffect an eight-wire cable. Since the lower end of the able was concreted into the base to a depth of 1 ft. 5 in. o form an anchorage, it was necessary that the cable luct should start from this level, which was, in fact, elow the top of the base slab. As the vertical core ormer could not be pushed down hard against the lower rille, owing to the change from square to circular ection, a junction sleeve was made up on site from 22 g. netal, and, as shown earlier, the 1 in. diameter rod used or forming the grout hole leading to the base of the cable was passed through this sleeve.

The normal type jacks were used, in a vertical position, nd no difficulty was experienced in stressing these ables. The extension of the wires was about $2\frac{1}{8}$ in., in a able approximately 36 ft. 0 in. long.

(c) *Circumferential Cables.* Four wire cables were first made up, their length being sufficient either to pass right round the outside of the tank walls, or half way round, depending on whether they were to be used for single-point or two-point stressing.

The finished cable was to consist of eight wires, so the four-wire cables were therefore made up in such a fashion that when combined they formed a flat eight-wire cable.

Cable 1 was first placed in position and stressed, then cable 2 placed over it and stressed in its turn. Small clips of 18 g. strip spaced at approximately 10 ft. 0 in. centres were used for binding the cable.

For supporting the cables in position on the tank walls prior to stressing, hangers were made up from 20 g. strip. These were hung from rawlbolts set into the top of the wall, and were notched at the correct pitch to carry the cables. Cable one was placed in the notch, and cable two was then later hung from its top pair of wires, which were placed in the notch above cable one.

Single point stressing was used for the majority of the cables, and consisted, in effect, of pulling the two ends of the same cable apart. The extension obtained on a cable 160 ft. 0 in. long was about $5\frac{7}{8}$ in. dependent on the friction between the cable and the surface of the tank wall. It was found that if members of the stressing gang moved along the cable, whilst the tension was being applied, tapping it with club hammers, the extension was noticeably increased, and this procedure was, in fact, adopted as a routine drill. A small pneumatic vibrator clamped on to the wires was tried out in an effort to overcome this problem of frictional resistance, but was not successful owing to its lightness.

Two-point stressing was used on the ring beam, and near the base of the walls, where it was necessary to ensure a more even distribution of the prestressing force. In this method, two jacks were used, both working at diametrically opposed points of the tank, on the same cable which only passed half way round the perimeter of the tank.

In order to achieve the most satisfactory conditions of tensioning, the aim was to obtain similar extensions at each jack, and this necessitated careful work on the part of the stressing gangs to ensure that the load was taken up equally on the two jacks. Since the jacks were diametrically opposite each other, field telephones were provided for intercommunication between the two gangs, and were found to be most effective.

The technique of tapping the wires during stressing was also employed, but was less necessary, owing to the shorter length of cable being tensioned. The average extension obtained on a pair of cables, together 160 ft. 0 in. long was about $7\frac{3}{4}$ in.

Stressing points on successive cables were staggered in plan, to assist in obtaining an even distribution of stress round the perimeter of the tank and in order to avoid too sudden a loading. All of the type 1 4-wire cables were fixed and stressed before the type 2 cables were fixed, with the exception of the ring beam, on which both cables were stressed first of all to take up the thrust of the roof joists.

A special jack carrying two strongbacks was used. One strongback was fixed to the jack frame, and the others, mounted on the ram of a hydraulic cylinder, moved away from the former as the load was applied by means of a normal hydraulic pump. The jack was hung by chain blocks from scaffold cross trees fixed close to the tank wall, so that it could be easily raised or lowered.

(d) *Quantities.* The quantities of steel used in each tank were as follows :—

Mild Steel Reinforcement
10.1 tons (excluding tank roof)

High Tensile Steel Wire

Base Cables	3.05 tons
Vertical wall cables	3.65 ,,
Horizontal wall cables	6.55 ,,
Total per tank	13.25 tons

Grouting

In order to provide permanent protection for the stressing cables, the vertical and horizontal cable ducts were grouted up as soon as possible after stressing.

The grout used was made up from equal parts of cement and sand, with sufficient water to give a w/c ratio of 0.45. It was mixed in a Colcrete grouting pan, which gave a very smooth and consistent colloidal grout.

The grout for the floor cables was led in through the sloping hole at one end of the cable duct, using the corresponding hole at the other end as an air escape. The apertures round the sandwich plates were first buttered up with mortar to prevent an excessive loss of grout.

The cable ducts in the lower of the three layers in the base were grouted first, then the central layer, and finally

the top layer. This was to ensure that, if one of the shims separating the ducts had become misplaced, there would be less likelihood of a grout leakage from one duct to another causing a permanent blockage.

It was found that the Colcrete mixer itself delivered the grout under sufficient pressure to force it through each floor duct in one continuous operation.

The grouting of the vertical wall cables was carried out from the sloping grout hole formed at the base of the cable. In this case, it was found necessary to lift the Colcrete mixer up on to the scaffold, with a longer delivery hose leading down to the base of the wall. In order to ensure that the grout did not run out again when the delivery nozzle was withdrawn from the grouting hole, the following device was adopted :—

An 18 in. length of rubber hose was wired securely at one end on to a 12 in. length of 1 in. water barrel. The other end of the water barrel was then thrust into the sloping grout hole and packed in position with mortar. About two dozen of these were fixed in position before grouting was commenced. The nozzle on the delivery hose of the Colcrete mixer was inserted into the free end of the rubber hose, and fixed with a Jubilee clip, and the delivery valve was opened until grout appeared out of the top of the cable duct. The delivery valve was then closed, and the rubber hose bent double and wired in that position. This effectively cut off the flow of the grout, and the nozzle of the mixer could then be withdrawn and passed on to the next cable duct. After about an hour, the grout had hardened sufficiently for the lengths of barrel and hose to be withdrawn, and the grout remaining in these was easily dislodged by rodding and tapping.

The grouting was an arduous and dirty task, and it was necessary to equip the grouting gang completely with oilskin suits, sou'westers, boots and goggles. There is no doubt, however, that they considered themselves amply rewarded when, on one occasion, the delivery hose burst, and the general foreman was covered from head to foot in a rich creamy grout.

Final protection to the sandwich plates anchoring the floor and vertical wall cables was effected by encasing them in concrete. A coping was cast on the top of the tank wall, to cover the vertical sandwich plates, and a casing 8 in. thick was cast round the tank base to cover the horizontal plates. (Figs. 4 and 5.)

Gunite

The horizontal prestressing wires and sandwich plates were protected by coating the whole of the outside surface of the tank with a 1½ in. layer of Gunite. This was applied whilst the tank was full of water, in order to ensure that no cracking of the coat occurred on subsequent filling.

4 in. × 4 in. × 10g B.R.C. fabric was placed all round the outside of the tank wall, and fixed with tie wires to the horizontal prestressing cables. The gunite, which consisted of 1:3 mortar of cement and sharp sand, was then applied in two coats.

Filling and Testing

After all stressing had been completed, the tanks were filled and tested. The initial filling was carried out in stages in order to distribute gradually the strains in the concrete and prestressing wires. Water was pumped in to a height of 12 ft. 0 in., at a rate of 1 ft. per hour, and the tank was allowed to stand for six hours. The level was then raised a further 12 ft. 0 in., at the same rate of filling, and allowed to stand for another six hours, after which filling was completed to the final working level of 33 ft. 0 in.

Throughout the eight tanks, no serious leaks occurred, but it was found that on the first five tanks, each of which had been constructed with the vertical contraction gaps in the walls, damp patches occurred up the joints between these gaps and the wall proper. The damp patches varied in degree from some which were only visible under dry weather conditions to some in which a small seepage of moisture was actually noticeable on the outside of the wall.

A few similar damp patches also occurred on some of the horizontal construction joints between lifts on the first two tanks.

In all cases where there was any evidence of dampness or moisture on the outside face of the wall the tank was drained, and a patch put over the corresponding area of inside face. This was done by hacking the area to a depth of about 1 in., to leave a thoroughly clean and well-roughened surface, and then applying a three-coat rendering of Sika, and it was found on retesting to be effective in every case.

The last three tanks, in which each lift had been constructed as a full circle with no gaps, required no remedial work whatever—*i.e.*, the 9 in. concrete walls were completely impervious to water under a head of 33 ft. 0 in.

Roof Construction

The Ministry specified that the roof should be a dome with a minimum thickness of 5 in. designed for a superimposed load of 30 lb./sq. ft.

The roofs could have been constructed in the orthodox manner with timber shuttering. This would have delayed the roof construction by several weeks as scaffolding and shuttering to the roof of each tank could not have been commenced until after the tanks had been tested. Furthermore, a considerable amount of timber would have been required owing to the limited time allowed for erection and the waste which is inevitable in circular cutting of timbers to shutter such curved surfaces.

As a precast concrete manufacturing yard was available, it was decided to form the roof with light precast concrete channel units supported on 16 6 in. × 4½ in. R.S.J.s curved to the shape of the dome and meeting at a central crown ring like the ribs of an umbrella. In this way it was possible to eliminate complicated and expensive shuttering and to cast and mature the roof units during the construction of the walls.

Whilst erecting the steel roof joists, the crown ring was carried on a central scaffold tower; the joists were not fitted into position until the vertical stressing of the tank wall and the circumferential stressing of the cables on the ring beam had been completed, and not until all circumferential stressing had been completed was the central scaffold tower removed, and the thrust from the roof joists transferred to the walls.

In conclusion we would like to express our appreciation of the help, assistance and guidance, both during the construction of these tanks and during the preparation of the paper, which we received from the undermentioned :—

Sir Charles J. Mole, M.V.O., O.B.E., F.R.I.B.A., Director General of the Ministry of Works; E. H. Bate Esq., M.B.E., M.C., A.M.I.C.E., Chief Site Control Engineer of the Ministry of Works. and the following members of Messrs. Stressed Concrete Design Limited; J. Singleton Green, Esq., M.Sc., A.M.I.C.E., A.M.I.Mech.E., M.I.Struct.E., J. W. Ager, Esq., B.E., A.M.I.C.E., Assoc.M.A.S.C.E.; R. F. T. Kingsbury, Esq.

Electricity Generating Stations*

By B. A. E. Hiley, M.I.C.E., M.I.Struct.E.(Member of Council)

A consideration of the development of electrical power production, tracing the history of generating stations from the earliest to the new Uskmouth 360,000 kW Station which is fully described

Synopsis

In this paper it is intended to survey generally the tory and development of the power station industry ich began in 1831 with a few simple discoveries and in relatively short space of time became one of the mary factors in all spheres of modern life and perhaps e greatest single source of motive power, industrial and mestic. It is a tribute to all those who have shared in ilding up so great an enterprise that practically all sential processes have made themselves dependent on electrical energy. There is hardly a corner of the rld so remote as not to have come under its influence. ngineers can look back on the glory of past achievents with pride, and forward with confidence to the most superhuman efficiency of the first atomic station. skmouth Station, with many others now building, is a od example of the extent to which practice has llowed theory in our own day.

Introductory

Michael Faraday discovered mechanical means of oducing electricity in the year 1831. This was llowed by other discoveries such as Wheatstone and ooke's electro magnets in 1845, Grammes ring wound matures in 1870 and so on. In other words, a dynamo as created, but efficient mechanical means for driving his had still to be found. Helped by Savery & Newommen's experiments, James Watt and his partner olton produced a steam engine provided with a conenser and steam jacket at his Soho Works, Handsworth, ear Birmingham. Watt's first practical steam engine was installed at the Broseley Ironworks in Shropshire in 1776. Agricultural machine manufacturers adopted the idea and such firms as Clayton & Shuttleworth, Fowler, Ransome, Sims & Head and Rodney produced steam engines of the portable or semi-portable type, whose successors are still found up and down the land at country fairs. The world-known firm of Babcock & Wilcox were also early in the field—indeed they installed the steam plant at the Holborn Viaduct Station, claimed to be the first steam station in this country, if not in the world, in the year 1882.

The third component required was lamps. Earlier lighting was by Arc lamps but the incandescent lamp came with Edison and Swan's inventions in the year 1880.

These early stations were used solely to provide electric light and were small, and installed in wooden huts or houses. Records show that it was at lighthouses such as Blackwall (1857), South Foreland (1858) and others until the year 1871 that these plants were first used. Some years later, in 1878, a French firm lit the Place de l'Opera, Paris. The Gaiety Theatre, London, was illuminated the same year. The Metropolitan Board of Works took an interest and employed a firm to light the Thames Embankment, an area outside the Mansion House, Billingsgate, and the river front between Waterloo and Westminster Bridges. Again, THE TIMES office had lighting installed as also Schoolbreds, Woolwich Arsenal and the London Bridge terminus of the L.B. & S.C. Railway. The provinces were not far behind. Plant was installed at Glasgow, Perth, Bristol, Newport (Mon.), Avonmouth, Liverpool, Blackpool and many other places, but it is difficult to find their chronological order.

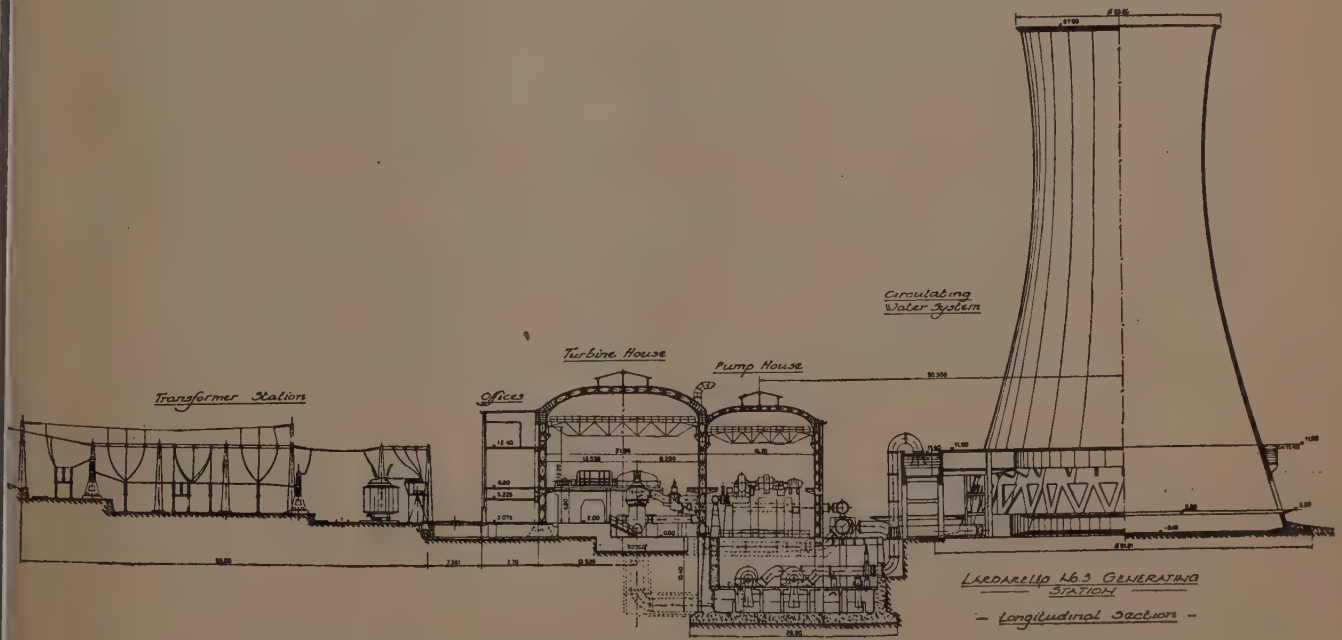

Fig. 1.—Geo-thermic power station—Larderello No. 3—Tuscany, Italy

**Paper to be read before a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, January 22nd, 1953, at 6 p.m.*

From the structural point of view it is worth recording that a contractor engaged on the construction of the great steel bridge spanning the Severn at Lydney used electricity to facilitate his work. Thus, even at this early date, a progressive contractor had his eye on new plant to aid his daily tasks.

Types of Stations

The sources of power used as prime movers in producing electricity are numerous. Coal, water, geothermic power (*i.e.*, energy taken from the depths of the earth), oil, peat, wind, sewage gas have all found a place in the electrical age. Now we may confidently look forward to gas turbines and atomic power.

Hydro-Electric

Hydro-electric stations play an important part in supplying power, and the number of installations is already considerable where fuels are scarce. They are divided into three categories—high, medium and low head. Notable examples of the first are several of the schemes associated with the Tennessee Valley Authority and Dixence, near Lausanne, which has a head of 5,750 ft., the highest in the world.

A large scheme is projected at Cariba Gorge on the River Zambesi on the borders of Northern and Southern Rhodesia. This particular scheme is interesting since at Wankie, in the same area, good coal is produced at 10s. per ton, thus affording an opportunity for economic comparison of the two methods.

In this connection it may be interesting to quote an article by the Vice President and Chief Engineer of the Pacific Gas and Electric Company which provides power to a very large part of the State of California. He says : " During the early years of the company when most of the power was generated in hydro-electric plants, the principal function of the steam plant was for standby use in the event of power failure, and to supplement hydro during peak hours. The more recent trend has been toward the use of steam power to carry large blocks of steady load. Also, many hydro-electric sites have been developed and only a limited number of sites now remain which can be economically developed. Therefore, the movement toward the construction of steam plant capacity is destined to become greater in future years. Ten years ago (1940) the steam plant capacity of the company represented 30 per cent. of the total installed capacity. To-day (1950) it is 42 per cent., and at the end of 1951 it will approximate 54 per cent. of the Company's total generating capacity." At the time this report was made that Authority was building three steam stations totalling 975,000 kW.

Low-head Stations, in the main, serve to top up steam stations or when a set is being overhauled.

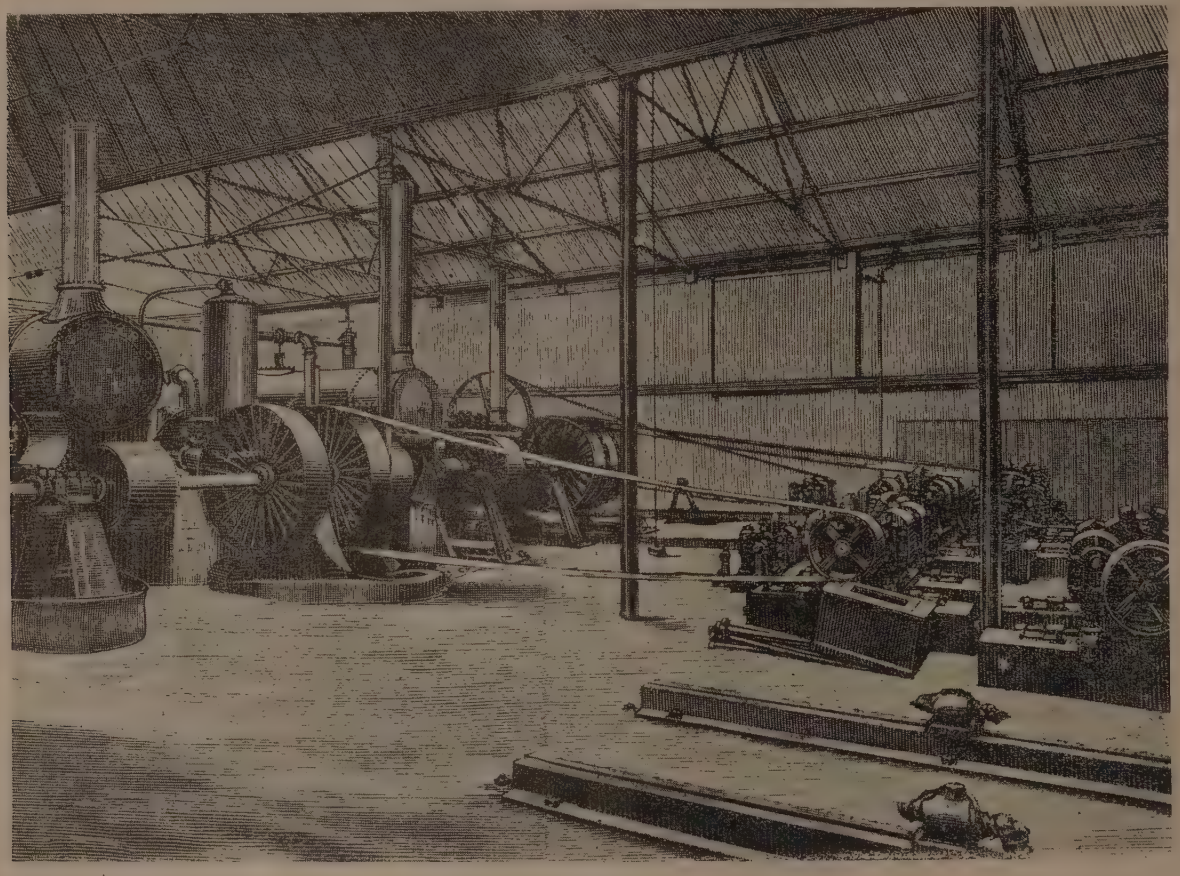

Fig. 2.—Brighton Power Station (1887)

In Britain the earliest hydro-electric station appears to have been built at Godalming, Surrey, in 1881. It was installed and operated by Messrs. Siemens Bros. & Co., under a yearly contract with the Town Council. The author of " Early days of the Power Station Industry," Mr. R. H. Parsons, writes : " Power was developed from a waterfall on the River Wey and the cables were laid in the gutters along the streets which were lit by both arc and incandescent lamps. The undertaking was an unprofitable one for the Company, and as a canvass of the citizens showed no prospect of a satisfactory demand, the supply was discontinued on May 1st, 1884, and gas lighting restored to the streets." The Société Edison of Italy, in making a similar claim for Milan, dates the installation as 1883, *i.e.* two years later than Weybridge.

Hydro-electric stations in Britain have, however, progressed but are generally confined to the mountainous districts of Scotland and Wales. With the schemes now

hand, hydro-produced power in 1956 will total approxiately 1,000,000 kW against 22,600,000 kW steam oduced. Of this one million kW the Galloway Scheme th its five power stations, nine dams, two barrages and ry considerable civil engineering works can contribute it 103,000 kW. Notwithstanding this the undertaking rms a very necessary and important link.

Hydro-schemes however are costly in the construcnal sense being about £200 per kW of installed plant as gainst £65 for steam generated power.

In Eire, however, where coal is almost non-existent, ere are several hydro schemes, the largest being the annon Station.

In passing, one should refer to the by-product of hydroectric stations. After the head of water has given up s power, it is controlled to a great extent and becomes vailable for land irrigation with a consequent benefit to

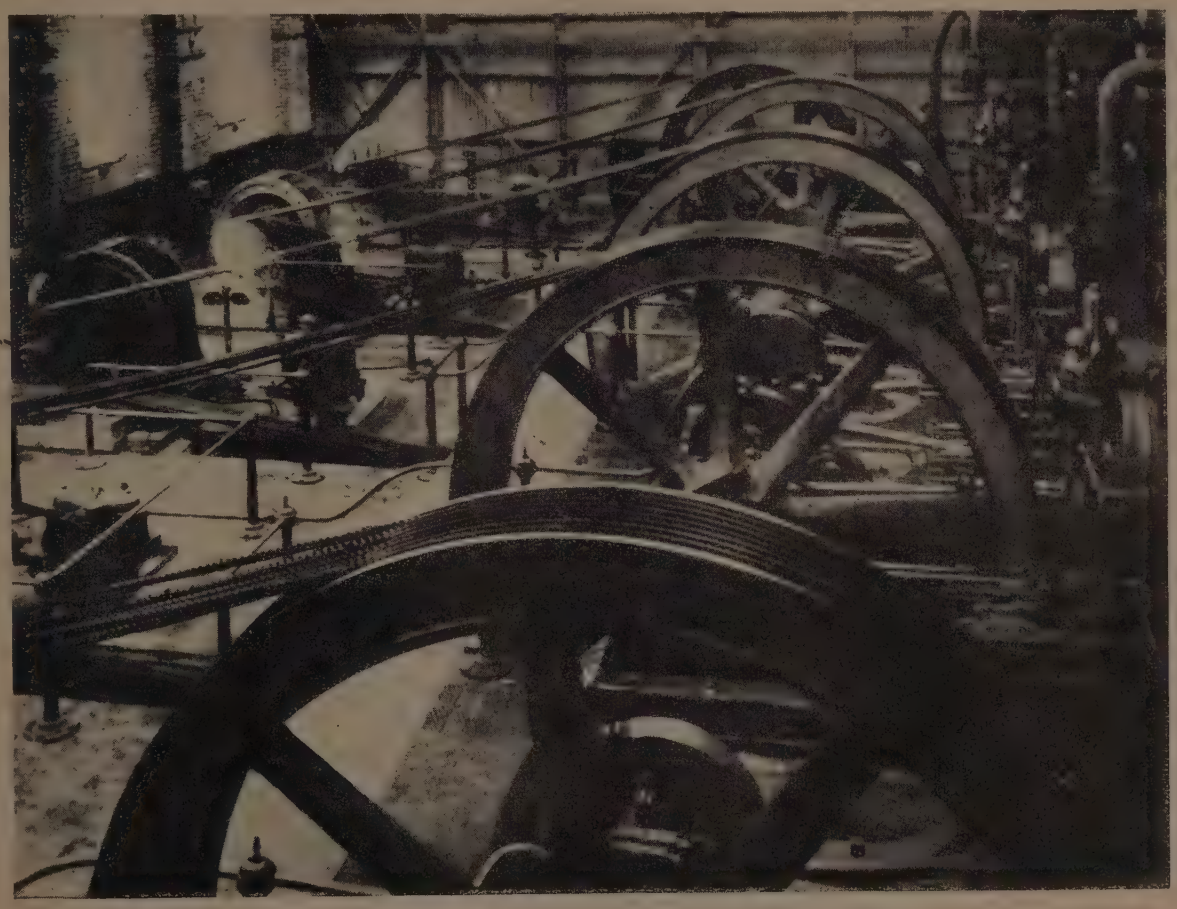

Fig. 3.—Llanarth Street Power Station, Newport, Mon. (1895)

griculture. The Tennessee Valley scheme has enabled a reat arid plain to be converted into a vast garden. In ssessing the benefits derived from a hydro-electric cheme, therefore, one should not calculate on the basis f electric power alone.

Geo-thermic Stations

Last session this Institution heard a lecture by Dr. Alfredo Mazzoni on the stations built at Larderello, in the Hills of Tuscany. Basically, the three larger stations already in operation producing some 130,000 kW, derive heir steam power from underground caverns between ,200 and 3,000 feet deep.

The late Sir Chas. Parsons believed that by deep drilling sufficient heat could be obtained to make coal burning unnecessary. Perhaps Larderello, where, it is contended, the heat is created by radio-active materials in the earth's crust at no great depth, is at once proving his theory and surpassing his expectations. It is believed that underground water gathers in large caverns and the consequent disintegration of the rock produces not only high temperature (284°F. to 419°F.) and high pressure (70 to 380 lb. per sq. in.) steam, but also an admixture of valuable chemicals such as boric acid and rare gases such as argon and helium, which are reclaimed as by-products of the power production. A French engineer, Raynault, in 1894 used the gaseous vapour of the soffoni to preheat the water of a small multi-tube boiler operating an 8 h.p. reciprocating engine. In 1904 Prince Conti actually produced electricity sufficient to light five electric lamps and this may well have constituted the first geo-thermic station. Later, Prince Conti experimented further and in 1930 he visited Hams Hall "A" Station with the object of finding a circulating water cooling system to meet the requirements at a proposed station at Larderello. He planned to tap the subterranean steam by pre-determined drilling and use it to drive Ansaldo steam turbines cooled by means of Hyperbolic Cooling Towers. So successful has he been that 130,000 kW of power is now being produced from the three main stations using nine Hyperbolic Cooling Towers. Plans for installing plant for a further 120,000 kW in this area are under consideration. In October last, four drillings 1763, 1689, 1339 and 3874 feet produced steam at the respective rates of 191, 106, 191 and 54 tons an hour at temperatures of 428, 381, 392 and 336° F. and 86, 62, 68 and 78 lbs. per sq. in. pressure. These came from 19·5-inch bores. The chemical or by-product side keeps pace with the electrical developments; the main by-product being almost pure boracic. Fig. 1 shows the latest completed station which consists of tappings in the ground, heat exchange

units, engine house and pump house, circulating water system and switching station. Compared with a coal-burning station there is a saving in constructional costs of rail sidings and marshalling yards, boiler and auxiliary houses, fan and gas houses, chimneys, electro-filters, coaling and ashing systems. The author believes that Larderello may later come to be broadly considered as the first atomic station since, apart from the heat developed relatively near the earth's surface, there appears to be a disintegration of rock material in process which may well be due to nuclear fission.

The work at Larderello is attracting much attention from districts where similar conditions prevail, particularly New Zealand and America. It is of structural interest to note that tubular steel construction was employed owing to the difficulty of transport and the general isolation of the area. Prefabricated in large

Peat-burning Stations

Two notable stations have been built in Eire where coal is practically non-existent. These are Allanwood (40,000 kW) and Portarlington (25,000 kW). The success of such stations depends on the efficient collecting of peat in sufficient quantity, drying it and feeding the boilers. A very ingenious mass cutting, drying and feeding plant has been devised and is working satisfactorily. Otherwise, these stations generally follow the lines of coal-burning stations. Hyperbolic cooling towers deal with the circulating water, and two such towers are in operation.

Sewage Gas Stations

Under the late D. M. Watson, M.I.C.E., the Birmingham, Tame and Rea District Drainage Board devised many years ago a method of collecting sewage gas which

Fig. 4.—Summer Lane Power Station, Birmingham (1906)

easily handled sections, the components could be taken to the site and welded into complete stanchions and roof trusses, the latter engaging tie-rods.

Oil Fuel Stations

In Britain the only large publicly owned station of this kind is now nearing completion at Bankside, London. A coal station was objected to on many counts; coal handling, ash disposal and damage to the fabric of London's historic buildings being the principal. In due course, no doubt, there will be sufficient reliable data to enable a comparison of costs to be made with the more orthodox steam stations. What is clear already is that in a station of this type fuel can be handled as in an ocean liner; it will be cleaner, occupy less space and will have no residues to dispose of. Cooling at Bankside will be effected by circulating Thames water.

enabled sufficient power to be generated for use on their extensive sewage works. No doubt others exist but must necessarily remain but small producers. However, this shows the ingenuity of man.

Wind Power

Many small units came into operation when power from the main sources of public supply was curtailed after the war. Hardly ranking as power stations these installations helped in difficult periods. However, the development of wind power for electricity generation has been dealt with by the Electrical Research Association and Mr. E. N. Golding, M.Sc., M.I.E.E., has provided us with a peep into the future.

It is claimed that a machine for the British Electricity Authority will generate at 100 kW. It will be erected near Aberdaran in North Wales and will consist of a

)wer 97 feet high and an air turbine located at base of)wer. The air is drawn through the turbine and up-ards through the tower by the rotation of the propeller nd is discharged through slots in the tips of the ropeller blades.

Good progress has also been made on a 100 kW. machine) be installed in Orkney for the North of Scotland ydro-Board and it is undergoing preliminary trials.

It will take time to perfect such devices, but a survey of ne conditions is being made in many countries.

When proved they will contribute but little to bulk upplies, but at the same time will make an economic ontribution to supplying power in out-of-the-way istricts where transmission from steam on other stations nay be costly. It has been estimated that building osts would be around £50 per kW and generation cost .25 pence per kW hour against 0.4 pence for steam enerated power.

Tidal Power

Whilst there are large sources of power obtainable from he tides, great difficulties exist to prevent its develop-nent. This method of producing electricity remains a

Fig. 5.—Princes Power Station, Nechells, Birmingham (1922)

challenge at the present, but schemes have been prepared and some day may be carried out.

Stations of the Future

In 1946, when many people visualised the extensive use of atomic energy in power stations, the author, conjectured the outline of one of the new stations, assuming that atomic energy could be counted as heat energy. Structurally it appeared that apart from atomic pile housing the new stations would shed the boiler house, electro-filters, chimneys and the whole of the coaling side, leaving a station somewhat as seen at Larderello.

Coal-burning Stations (1882-1952)

In Britain, practically all electric power is derived from coal-burning stations, so it is interesting to review their history.

Holborn Viaduct Station, London—1882.

This station is reputed to have been the first public power station in this country, if not in the world. It was located at No. 57 on the north side of the viaduct. The plant consisted of an Edison dynamo coupled direct to a Porter-Allen horizontal steam engine mounted on a base plate, the whole weighing some 22 tons. No great structural problems here—just a jobbing builder's work. The water tube boilers were constructed by Messrs. Babcock & Wilcox.

Legislation and Development

Four years before the Holborn Viaduct Station was operating, Parliament became interested in electrical supply problems. Gas and water undertakings worked under statutory powers granted by the Board of Trade, but not so the electricity supply undertakings which were individually so tiny as to be confined to lighting of theatres and small areas as described. However, companies and certain municipalities considered the potentials of the situation. Thirty-four applications were made for statutory authority to supply areas in the year 1878. A Commission of Enquiry was set up in 1879 and their findings were that : " In considering how far the legislature should interfere in the present conditions of electric lighting, the Committee would observe generally that, in a system which is developing with remarkable rapidity it would be lamentable if there was any legislative restriction calculated to interfere with the development." Parliament did not act on that recommendation, but in 1882 Joseph Chamberlain's Electric Lighting Bill was introduced and became law. This Act was not popular with potential developers. It provided that the Board of Trade should be empowered to grant licences to local authorities or private companies with the consent of local authorities to establish electricity undertakings in defined areas. The licences were to last five years but were renewable. The Board of Trade were also empowered to grant provisional orders to local authorities, or to private companies without consent of local authorities, but these had to be confirmed by Parliament. Local authorities were empowered to purchase undertakings at the end of 21 years or at the end of every seven years thereafter, at fair market value but there was no consideration for goodwill.

This Bill satisfied no one and, indeed, hindered progress of the industry. However, an amending Act was passed in 1888 following attempts by Private Members Bills to rectify matters. It gave a fresh impetus and stations were built all over Britain. The Act's most important conditions were : —

(i) Local authority's consent should be obtained before granting a provisional order.

(ii) The Board of Trade to have power to override any unreasonable refusal by a local authority.

(iii) The Board of Trade to be empowered to vary terms of sale so that the sale may be negotiated.

(iv) The period of security of tenure increased from 21 to 42 years with options increased from seven to ten years.

With the growth of supplies some control over the activities of authorities and companies became necessary and this was effected by the 1909 Act.

For instance, the Board of Trade could now enable any authorised undertaker to acquire land compulsorily for a generating station even outside his area of supply.

As early as 1900, Private Bills began to be presented to Parliament seeking powers for authorities to supply large areas. The South Wales Power Distribution Company, for example, obtained power to serve an area of 1,050 sq. miles; the Yorkshire Electrical Power Company an area of 1,800 sq. miles, and so on.

The 1919 Act centralised control still further by setting up "Electricity Commissioners" to co-ordinate generation and main transmission on a voluntary basis.

A further Act passed in 1922 was designed to facilitate Commissioners' efforts of co-ordination.

The important 1926 Act was based on the "Weir" Report for concentrating the generation of electricity in a comparatively limited number of large and efficient stations all interconnected by a high-voltage "grid" system. It established the Central Electricity Board, whose main duty was to consider what existing or projected stations should rank as "selected stations," and brought the "grid" into existence. Owners of non-selected stations could be compelled to shut down and take supplies from the "grid" on certain terms. At that time there were 440 power stations belonging to authorised undertakings.

Finally, on April 1st, 1948, the new British Electricity Authority with its 14 Area Boards took over the whole of the generation and distribution of electricity in Great Britain under the general direction of a Minister. The Authority is required to promote economical methods, development and research, cheapening of supplies and standardization of methods, equipment and supply. It also administers a reserve fund for the prevention of tariff fluctuations. Wide powers were given to the Authority to act in matters of its concern, ranging from the manufacture of equipment to the advancement of the skill of its employees.

Consultative Councils in each Area were also established.

This reference to Legislation is important since progress was everywhere affected by it.

Brighton "B" Power Station—1887 (Fig. 2.)

Situated in the centre of the town in Gloucester Road, the station buildings were of brick and measured 70 ft. long by 55 ft. wide. Light stanchions and roof trusses were in steelwork supporting a slate roof lined in timber. The floor was of concrete and the boiler plant consisted of three semi-portable boilers as shown. It is assumed that these agricultural types of boilers were driven in, jacked up, wheels removed and boilers anchored on a concrete foundation. The dynamos were mounted on slotted metal bearing-pieces to enable the tension in the driving belts to be adjusted.

Llanarth Street Power Station, Newport—1895 (Fig. 3.)

Here can be seen John Fowler boilers driving Uskside Engineering Company's generating sets with ropeways,

Fig. 6.—Hams Hall "B" Power Station, Birmingham (1942). Interior of engine house

as well as the familiar structural design of the earlier power stations; heavy brick walls relieved by arches all in glazed brickwork of two or three colours. One can visualise strip foundations in ordinary concrete of no particular specification. Also noticeable is the temporary end that seems to be necessary even to-day, unless the complete station is built as one contract.

The Town Council obtained first-hand information inspecting stations at Chelsea, West Brompton, etc., and applied for a provisional order which was granted by the Board of Trade in 1891.

Mr. Robert Harcourt, M.I.E.E., was appointed Consulting Engineer and tenders for buildings, plant etc., were requested in October, 1893. Sixty firms tendered, Wm. Gradwell of Barrow-in-Furness obtaining the Civil Engineering Contract.

The station was closed in 1928 after 33 years' service, d is now used as a sub-station.

Summer Lane Power Station, Birmingham (Fig. 4.)

This station when it was built in 1906 was considered e super power station of the country. Here are seen)t semi-portable boilers but nine Bellis & Morcoms ciprocating engines directly coupled to the generators, id two small 500 kW Parsons turbines. The total pacity of this station in 1906 was 11,500 kW. Here ain will be noticed the familiar heavy bearing walls and lieving arches on a brick-concrete foundation support- g an overhead gantry. Glazed bricks with the usual)lour relief were used internally. The cooling system as closed with the nearby canal arm until the capacity of ie station was raised, ultimately reaching 50,000 kW by)26, which necessitated installing a large timber cooling)wer and several Worthington cylindrical towers of the ietal-filled type.

rinces Power Station, Nechells, Birmingham—1922 (Fig. 5.)

Following the implementation of the Parsons patents, eam turbines took the place of reciprocating engines

Fig. 7.—Uskmouth Power Station—Flooding of high tide, September 5th, 1947. +22.50 O.D. Newlyn. (N.B. Basement Level of Station is +6.00 O.D. Newlyn)

nd here is seen the combination of steam turbine and enerator in one of the earlier stations. This station ventually had 10 No. turbines with a total capacity of 62,000 kW. With larger outputs, the circulating water ystem became of major importance. This station em-)loyed some 45 timber cooling-towers circulating ',800,000 g.p.h. and special care had to be taken to)rotect them from fire. As an instance of present-day levelopment, three hyperbolic cooling-towers are now ised to circulate 9,600,000 g.p.h. at the new Nechell's " B " Power Station. The buildings have piled foun- lations and structural steel superstructure.

General picture of the engine house of the Hams Hall " B " Power Station at Birmingham built during the war (Fig. 6.)

The difference in the structural treatment seen here is further developed in the station at Uskmouth which will now be considered. Incidentally, Ham's Hall " B " Station is a complete ferro-concrete structure clothed in early English brickwork.

British Electricity Authority

The installed plant of the British Electricity Authority on December 31st, 1951 is given as 15,737,000 kW. It is planned to add a further 8,000,000 kW by the end of 1956, which calls for an accelerated programme averaging 1,600,000 kW per annum and making a total of installed plant of nearly 24,000,000 kW by that year, assuming that existing plant is maintained.

Of this total, hydro-electric plants of high and low heads may account for something like 1,000,000 kW., whilst the oil-fired station now being built at Bankside, London, will produce a further 120,000 kW. It will therefore be appreciated that by far the major part of the total output is by means of coal-burning, or thermic, stations.

Uskmouth Power Station (1948-1952)

Site

With a coal-burning station the first consideration is to find a site which, whilst having reasonable access to coal and large volumes of water, offers also a method of ash disposal. If the station is to take water from a river or waterway, the fisherman, whose ancient rights are sacred to the hearts of River Commissioners and others, must be safeguarded and satisfactory guarantees given that the water will be returned reaerated and cool.

The Ministry of Town and Country Planning must also be satisfied that the development of the countryside will not be prejudiced in any way and the Royal Fine Art Commission must see the architectural treatment. Only then may the British Electricity Authority proceed with their plans and endeavour to obtain the land. In an organised community all these processes are matters of importance. Nevertheless, they are tedious and often cause lengthy delays, particularly when approval becomes the subject of an official inquiry. Such was the position at Lincoln, Durham and Newport, to mention only a few.

Mr. T. H. Wood, M.I.E.E., Borough Electrical Engineer and Manager in 1948, discovered the Uskmouth site and initiated the scheme which, on April 1st, 1948, was taken over by the newly-formed British Electricity Authority. Despite foundation difficulties common to the east bank of the river, this site at the confluence of the River Usk with the Bristol Channel offered a very large expanse of relatively flat agricultural land (part of which had been " saltings ") with ample water supplies and a pilot rail line laid by the old G.W.R. when they wished to promote industrial development there. Incidentally, this mile or so of dead straight rail became the base line on which the grid survey was made. The site acquired will be sufficient for both the present "A" and the projected " B " Stations.

Ample accommodation exists for rail marshalling yards coal store and (by reason of the existing ground level) for the vast quantity of boiler ash. When one remembers that the ash content is approximately 20 per cent. and the consumption of coal on the "A" Station alone averages some 3,400 tons a day, it can be realised what an important part the selected site must play in its disposal. In passing, it may be recorded that the fly ash, a by-product of the station, may ultimately become a useful building material. Experiments in the Midlands indicate that bricks can be made successfully.

Fig. 7 shows the site in flood, the target poles indicating the width of the station. In addition to the land survey which carefully picked up all details of the irrigation system, the flood gates, river and defence lines, it was necessary to make a soil survey as well as a survey of the river and tides, remembering that on this coast the normal tidal range is supplemented by additional heights when there is an on-shore wind and a falling barometer. These factors combined can cause an additional 3 feet, producing a total range of 44 feet.

The soil mechanics report showed that the site would have to be piled and, further, that piling would not be easy.

Road and Site Levelling

Negotiations completed, permission was obtained to enter the site on March 20th, 1948, when a pilot contract was let. This contract covered preparation for levelling, drainage and access roads, the extensive railway embankments and the building-up and drainage of ground to be used as a coal store, all of which entailed the use of some half million tons of red ash and quarry waste. The only problem here was maintaining the reens and drains established over the centuries by the farmers who, by means of earth banks and primitive wooden sluices, kept the land free of sea-water and yet conserved fresh water for cattle-raising and cultivation.

Wherever new embankments crossed these long-established features, the latter were maintained by culverting—a simple precaution that paid handsome dividends in subsequent site development.

Only such areas as were immediately required to be raised were filled and consolidated, since the station ash will in time completely cover all low ground not wanted for the station buildings, rails or roads.

Camp Buildings and Amenities

In view of the estimated peak labour force of some 2,500 men, a camp was designed to give sleeping accommodation, ablution and drying huts, recreation and catering accommodation. This was built on a unit principle but of such strength as to combat the gales and hard weather on this exposed part of the coast. The camp is administered by the Authority and is well patronised. It is an undoubted amenity much valued by all who work on the site. A cine film record of progress has been made and is often shown. The construction was of Ministry of Works type precast concrete cantilevered span trusses with brick infill and asbestos cement roof with ridge ventilators. Concrete floors suitably surfaced with a bituminous solution gave a fine and easily cleaned surface. It should be remembered that the site is isolated, and little or no ordinary living accommodation is available inside or around Newport.

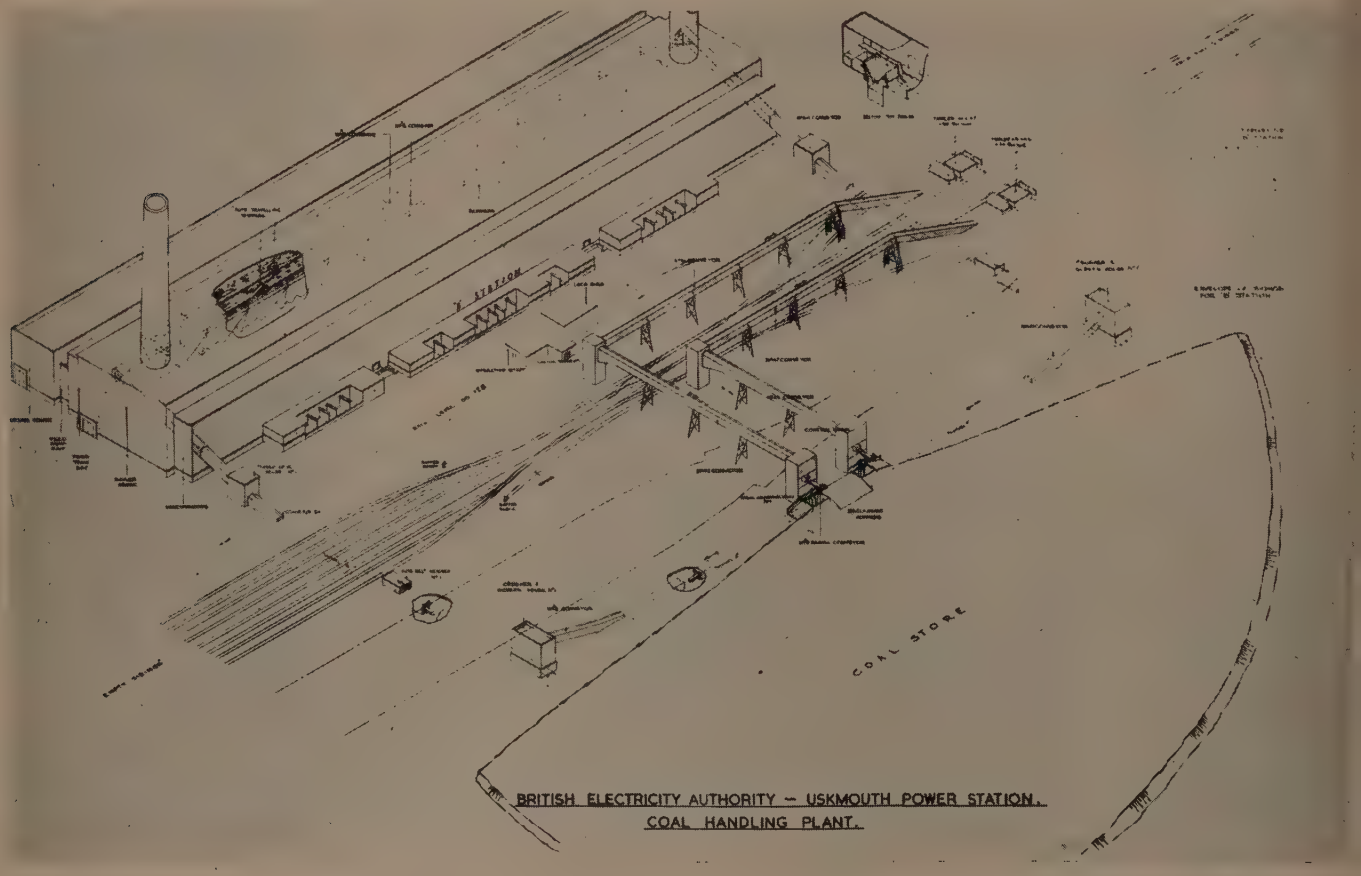

Fig. 8.—Uskmouth Power Station. Layout of coal-handling plant

Coal Handling, Storing and Railways (Fig. 8.)

In planning the full development of the site it was necessary to plan railway sidings and marshalling yards for the "A" Station and a possible "B" Station of equal or possibly greater capacity. The first station is on the

west of the peninsula and in line with the existing single track previously referred to.

Intake of coal for the "A" Station is approximately 3,400 tons per day which can be taken either from the large semi-circular store (capacity 108,000 tons, or five weeks' supply) or discharged direct from trucks on to two separate tipplers of the usual side-discharge type. If wanted directly in the station, the coal is conveyed from tipplers to crusher house and thence into the boiler house by conveyors at each end of the station, where it is fed into the concrete bunkers by trippers on twin conveyors. If the bunkers are full the coal can be readily directed to the coal store. Reclamation from store is by means of scrapers and a bulldozer which moves coal to an underground hopper at the centre point. From here it is taken through crushers and into bunkers as previously described. The tipplers, hopper and crusher housings, etc., are all ferro-concrete structures. The total conveyor capacity is 600 tons of coal per hour. The conveyors are housed in light steel structures clad in R.P.M. sheeting with segmental roofs. Foundations are piled and capped.

The eight miles of tracks forming the sidings required for the "A" Station will accommodate 480 ten-ton trucks of coal. There are sufficient sidings for marshalling "fulls" and "empties" as well as "crippled waggons." Simplicity has been aimed at.

The track ballast consists of Blue Pennant and limestone, graded from 2 in. down, carrying prestressed concrete sleepers with standard chairs. The turnouts and crossings employ timber sleepers.

Main Buildings, Piled Foundations (Fig. 9.)

Generally the plant foundations have been designed independently from the building structures. The main items consist of six heavy turbo blocks measuring 62 ft. 0 in. long by 18 ft. 0 in. wide and extending from basement level at + 6.00 O.D. to engine room floor level at +34.00 O.D. with condensers between the steam and electric ends. The turbines and blocks each weigh some 1,700 tons. The sets are 60,000 kW G.E.C. hydrogen cooled rotating at 3,000 r.p.m. laid out in "line ahead" formation. The circulating water ducts at the station are accommodated at the back of the retaining wall and in this way a large unencumbered basement floor is provided.

There are 12 Babcock & Wilcox boilers using pulverised fuel and generating steam at 950 lb. per sq. in. and 925°F. with steaming economisers, draught and milling plant. Each successive pair of boilers supplies steam to a single receiver. Ten of these boilers work and two are in reserve. The foundations are of suitably grouped piles which support 2,200 tons per boiler.

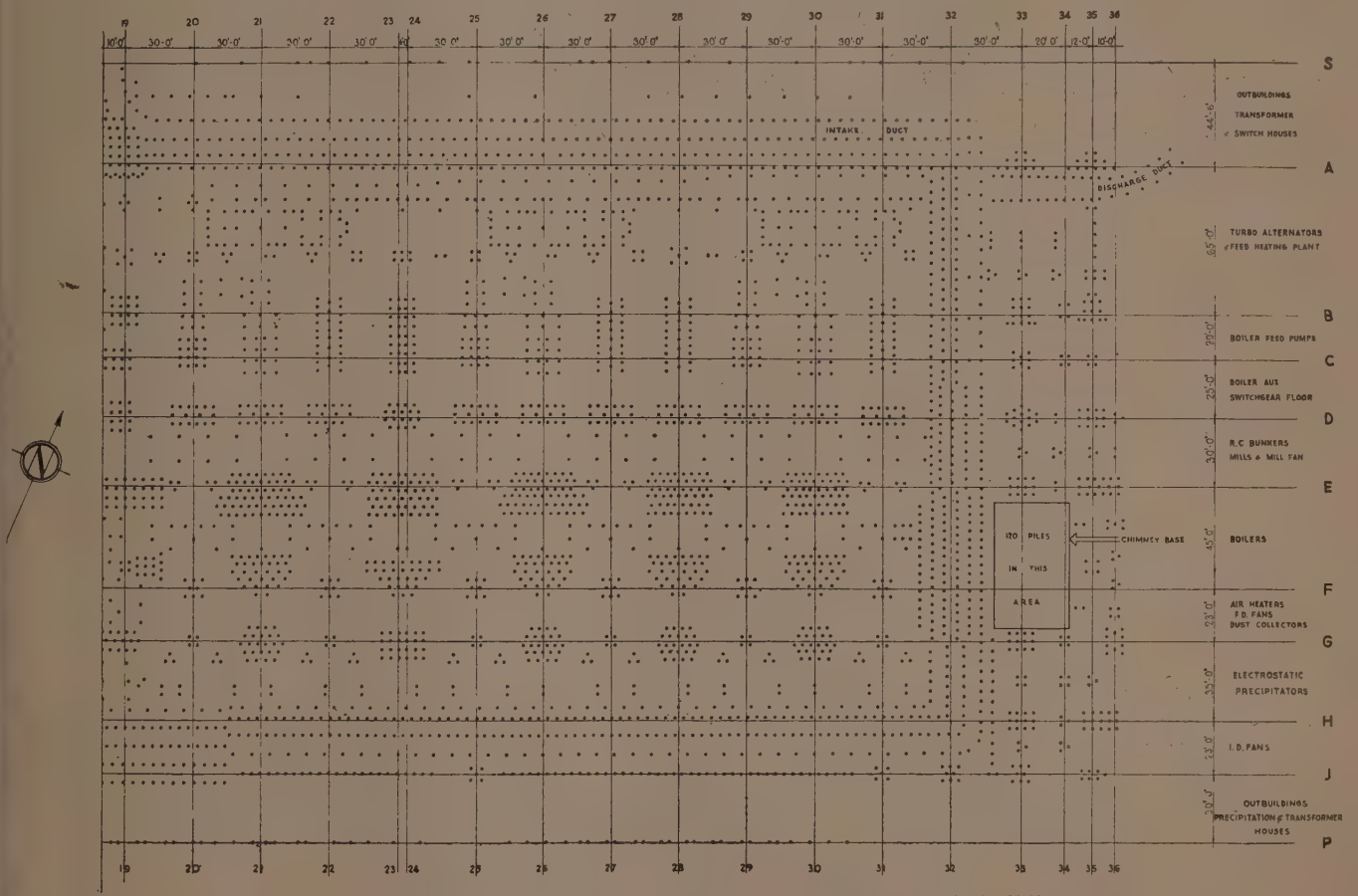

Fig. 9.—Uskmouth Power Station. Half-plan of piling for main buildings

The next important load comes from the 12 electrostatic precipitators each on piled foundations with loads of 400 tons per unit.

Centicell dust collectors and electrostatic flue gas cleaning ensure that grit emission from the two 300 ft. high ferro-concrete chimneys is kept to the minimum. This installation is obligatory.

The Ash Disposal Plant

Coal ash from the water-filled hoppers beneath the boilers is collected and crushed in a central ash pit below the station basement and then pumped to the ash field by means of three pairs of series-connected pumps. Dust from the precipitators is similarly disposed of.

This disposal plant, including crushers and pumps, is piled as for the rest of the station. The constructional

work was carried out within a steel sheet piled cofferdam and frames.

Piling

As anticipated, piling offered difficulties. Due to the inclination of the silt stratum toward the river, piles had to be driven into the marl. Water jetting from level +4.00 O.D. was first considered sufficient to help the piles through to the marl level, but after excavation there remained an average of 14 ft. 0 in. of estuarine clay over the sand which, being waterlogged and held top and bottom between the clay and marl, did not allow the pile to pass downwards with reasonable ease.

The 16 in. and 18 in. square precast piles were between 45 ft. and 75 ft. long, these latter being located at the loading bay areas and calculated from level +28.00 O.D.

ground to such an extent that ashes were required as a mat to enable work to be continued.

Seven modern driving sets were employed and records taken for every pile driven, so it was possible to check carefully what was happening. The 2 in. diameter jetting tubes in the piles when plumbed indicated what occurred below ground. With few exceptions the piles were not damaged at their lower levels and it was mainly the heads that spalled or shattered.

Driving was organised so as to offer an " open face " to the operations, though in certain cases where rakers were required to pass between vertical piles already driven for turbo bed bases this was not possible.

In the earlier driving, Oak, Greenheart and Pinkadoo dollies were used, the latter giving the best results (12 piles per dolly). Attention was then paid to packing

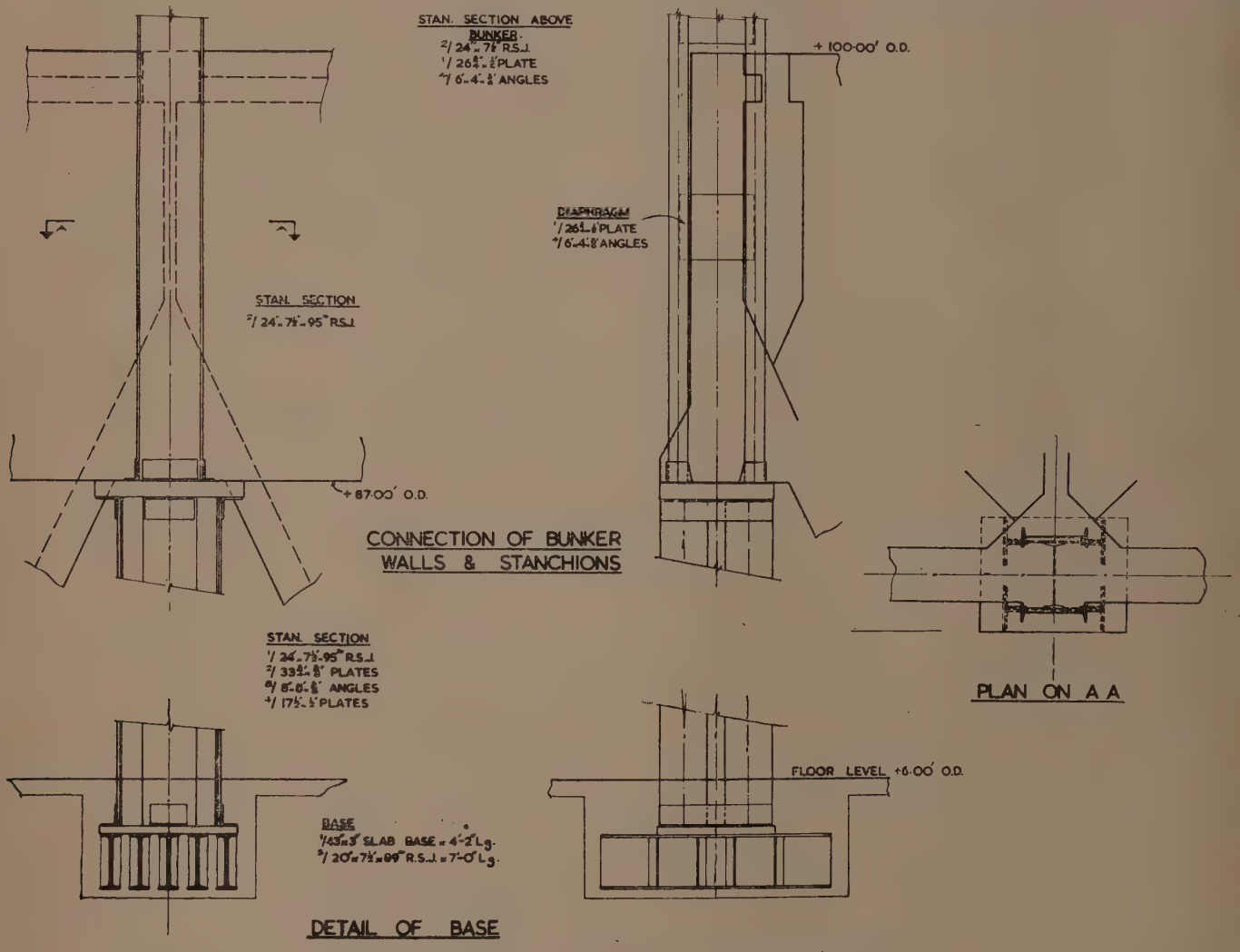

Fig. 10.—Stanchions between boiler house and bunker bay

It became obvious that to reach the marl some means had to be found to pass the sand barrier which was breaking dollies and doing no little harm to the piling plant employed. Such things as trigger arms to operate valve gear, gullick clips, flexible copper high-pressure jetting tube connections, ram casings and steam glands all gave trouble until a solution was found. Jetting pressures of 200 lb. to 350 lb. per sq. in. were tried but this only served to raise the pile—evidence of the " dead hand " effect of the overlying bed of clay and the unyielding marl. A slightly enlarged section where the toe joins the main shaft of the pile was tried in the hope that this would create a clearance, but the effect was that the pressurised water flowed upwards and liquified the top between top of piles and the dollies. After much experimenting, resin-bonded laminated timber dollies of some 92-ply (thickness 4 in.) capable of withstanding a uniformly applied pressure of 43,000 lb. per sq. in. were used successfully. Much better results were obtained and, although they cost £7 each, the unit cost per pile driven was decreased materially. Research continued and finally a plastic dolly costing £18 solved the problem of hard driving. Records exist showing that 40 piles were driven with one such dolly. This plastic material consisted of Bakelite reinforced with cotton fabric, the thickness being approximately 4 in. For 16 in. piles the dolly was 14 in. × 14 in. × 4 in. and for 18 in. piles 15½ in. × 15½ in. × 4 in.

It is pleasant to record that under these conditions of ıard driving the casualties, though high in the early ;tages, amounted to but 2 per cent. of the total.

The general piling scheme is the usual grouped layout or a large structure, but the chimney foundation piles ncluded in this contract were interesting. These piles ıad to penetrate the same silt band but at closer spacing.

This problem was solved by carefully driving $\frac{3}{8}$ in. :hick, 24 in. diameter, steel tubes down into the marl with a 4-ton single acting steam-hammer with a drop of ı2 in. to 18 in. These tubes were made of flat plate brought tó the site in convenient lengths and welded together. A steel helmet with packing and dolly was used and the cutting edge of the tubes stiffened. This was a delicate job splendidly handled. Once into the marl, the core was scooped out and the piles driven inside the tubes. The inter-space was then pressure-grouted. These piles penetrated the marl up to 5 ft. Apart from the special difficulties mentioned no other obstacles not normal to this type of work were encountered.

It should be recorded that at level +4.00 O.D. the contractor suffered little inconvenience from water, notwithstanding that tides only 250 yards away rose to +20 ft. on occasions. The contract did, however, provide for dewatering devices if required.

Chimney Structure

There are interesting points in the design and economics of power station chimneys.

It will be appreciated that large savings in cost and materials can be effected (a) by making the chimney an integral part of the boiler house, so that the main shaft commences at roof level, instead of a separate structure, and (b) by using ferro-concrete rather than brick construction.

The two Uskmouth chimneys have been designed on these principles, having fluted reinforced concrete shafts 206 ft. high rising directly out of the boiler bay and tapering from 34 ft. diameter to 25 ft. diameter internally at the top. The corresponding wall thicknesses are 10 in. and 6 in., excluding the brick lining.

Building Frame Foundations

As these were mainly separate, the level of group pile-capping could be determined in the contract plans without considering plant foundations. It is interesting to note here that in many cases where all foundations are not piled but rigidly connected, deep plant foundations may necessitate taking frame foundations considerably below an otherwise satisfactory stratum.

The building frame foundations in the engine house amount to approximately 500 tons per stanchion due to the heavy 175 ton overhead gantry used in installing the turbines and maintaining them in use.

Fig. 11.—Uskmouth Power Station. Interior view of engine house roof

Steelwork Superstructure

There can be very little that is new in the design of a steel superstructure for a modern power station, whether it be designed as a rigid or semi-rigid frame or to the B.S.S. 449-1948 with allowances made for wind moments at the stanchion connections. The final design was a combination of all three methods and, so long as structural engineers are forced by circumstances to accept certain methods of fabrication and erection offered, this must always be so.

The major erection problem to-day is the shortage of riveters and welders resulting in too frequent requests to use bolts, when even turned-barrel bolts are impossible to obtain.

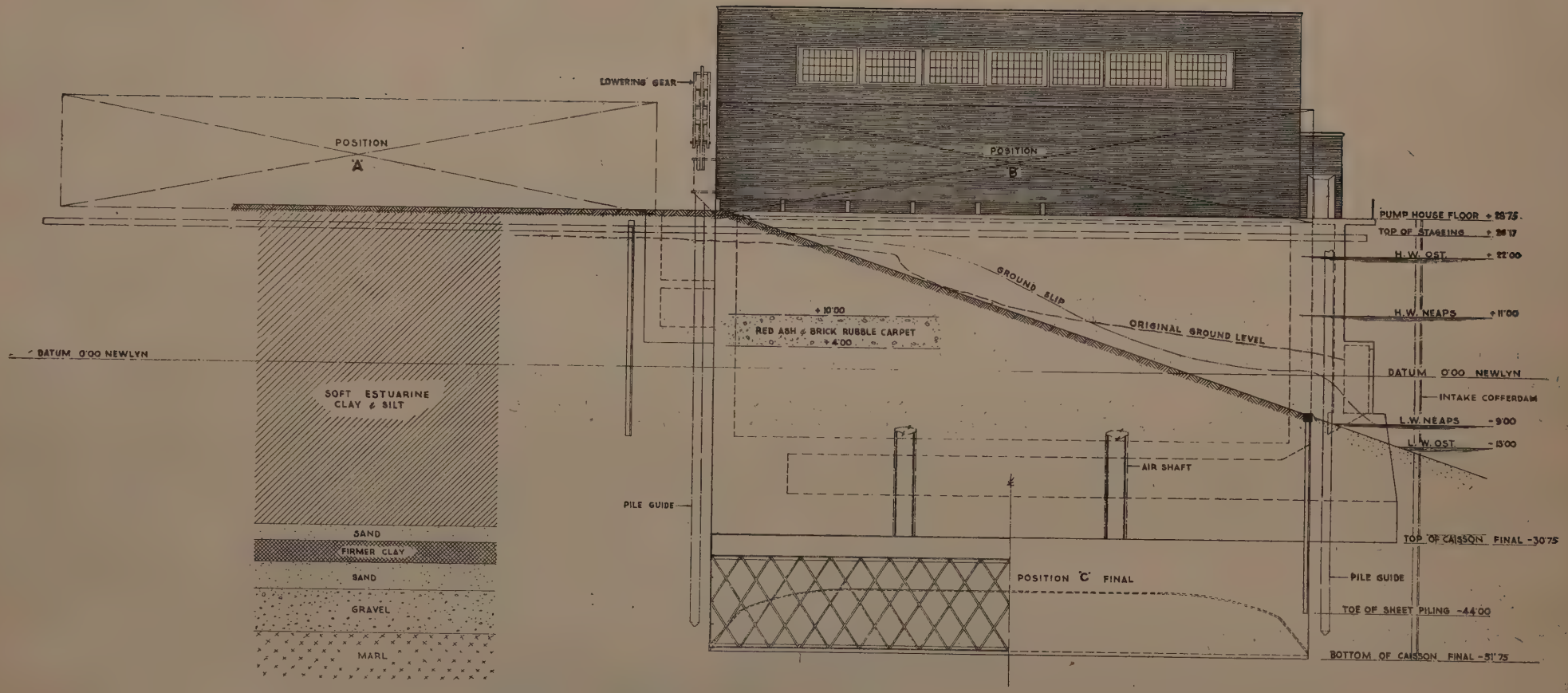

Fig. 12.—Uskmouth Power Station. River intake and pump house. Diagram of caisson sinking and ground conditions

The Uskmouth steelwork, maybe differs in one way rom previous power stations in that it incorporates rovision for air raid precautions. The compound eams at the windows are designed for a vertical load quivalent to the weight of a 13½ in. thick brick wall bove and also against blast.

The turbine house bay is 65 ft. 0 in. span with a crane ft of 175 tons. The stanchions are at 30 ft. 0 in. entres and are of a built-up section of joists, plates and ange angles with a continuous plate web. The flange entres are only 3 ft. 0 in. and this has removed the rails subjected to heavy local concentration of stress from wheel loads of up to 70 tons per wheel, including impact.

At the request of the British Electricity Authority (on the advice of the pump manufacturers) the electric pumps are supported on independent steel structures entirely separated from the main superstructure. The steam pumps, however, are supported on the pump floor steelwork and their supports designed to avoid harmonic vibration. The firing floor was generally designed for a superimposed load of 3½ cwts. per sq. ft., but in addition a point load of 5 tons was allowed over certain specified

Fig. 13.—Uskmouth Power Station. Latest progress photograph

ppearance of heaviness so often associated with in- ustrial buildings of this kind.

The turbine house floor is supported independently of ne turbo blocks and is designed for a superimposed load f 4 cwts. per square foot with specially loaded areas for ithdrawal of the rotors.

The roof girders for both the turbine and boiler house ays are of similar design, those in the boiler house being esigned to lift the boiler drums, and the turbine house nes to assist in erection of the 175 ton capacity travelling rane.

The problem of the expansion joint in the turbine ouse crane rails was given a good deal of thought both by ne engineers and the crane contractors, and this co- peration led to a solution which has been accepted by ritish Electricity Authority engineers generally as a tisfactory method of forming expansion joints in crane areas to cover loads due to plant erection. On the beams this superload was reduced to 2 cwts. with the same point loads.

This was a far-sighted policy as, many months after the commencement of the steel erection, the British Electricity Authority firmly stated that no part of the roof should be left open as access for delivery of the boiler structures from a crane erected on the roof steelwork. Hence large sections weighing up to 10 tons were carried across the firing floor over the specified areas, and a travelling gantry crane (capacity 5 tons) was slung from the underside of the boiler house roof girders. This was a further step to assist in the speeding up of the erection of the station. The boiler drums, weighing 48 tons each, were also taken across the basement floor on rollers and hoisted into position from the roof girders by means of lifting beams.

Again, the firing floor steelwork is independent of the boiler structure, the steel beams trimming the floor openings being capable of supporting the temporary loads from the boiler drums.

The reinforced concrete bunkers are supported on the thick cap slabs of the heavy compound stanchions of the bunker bay. These stanchions are formed of two joints spaced sufficiently apart from the reinforced concrete bunker walls to run continuously between. This detail is illustrated in Fig. 10 and is a good solution to the problem of two different materials being used to the best advantage.

The main features of the steelwork design are the steel structures supporting the reinforced concrete chimneys. Designed as rigid frames, the total dead load from one chimney is 1,500 tons with a wind shear of 80 tons. The calculated deflection was 0.90 inches. Much thought was given to the detailed connections which were site riveted.

The maximum load on the stanchions between bunker and boiler is 1,250 tons, the stanchions being at 60 ft. 0 in. centres. Small grillages were used under these stanchions taking advantage in design for the increased direct compressive stress allowed under C.P. 114/1948 on the 1 : 1½ : 3 concrete in the pile caps. A detail of this base is shown in Fig. 10.

The superstructure is approximately 872 ft. long and 268 ft. wide, consisting of turbine house, pump, tank, bunker, boiler house, precipitator and I.D. fan bays. There are two expansion joints in the length of 872 ft. (Total weight of steelwork = 15,444 tons).

Time from commencement of erection to completion: three years.

132 kV Switchouse Steelwork

The steelwork superstructure was designed as a semi-rigid frame with site riveted connections at the stanchions.

Steelwork around the contact breaker cubicles was designed to resist a blast pressure of 72 lb. per square foot.

Length and width of building = 460 ft. 0 in. long by 90 ft. 0 in. wide.

Total weight of steelwork = 1,080 tons.

Time from commencement of erection to completion : nine months.

Floors and Roofs

The floors throughout the station are of *in situ* ferro-concrete or open steel grid-work, while the roof is of pre-cast concrete beams of unit construction. These are hollow to assist in the prevention of condensation. Screeds to falls and cork under asphalt gives the necessary insulation.

Cladding

The exterior cladding is of a straw-thatch coloured brick with reconstructed stone caps and features. The fenestration is ample and with electrically operated opening gear ventilation can be easily regulated.

Ventilation

Fig. 11 is an interior view of the engine house roof, and shows the ventilation. RPM ventilators have been used throughout on this station.

Certain sections of the office block at the west end, including locker rooms, etc., are ventilated artificially.

Circulating Water System (Fig. 12.)

The circulating water system is of particular interest as consideration had to be given to the fact that the River Usk, from which the circulating water is drawn, has a tidal range of approximately 44 ft. The quantity of water required is 15,000,000 gallons per hour, the total flow at low water being 60,000,000 gallons per hour, with a period lasting a few minutes at the turn of the tid when there is no flow whatever.

Considerations of economy made it necessary t utilise the maximum effect of syphonic assistance, havin regard to the cost of excavation for the main building an the circulating water inlet and discharge ducts. Thes ducts are of rectangular section under the station base ment, joining twin circular ducts eight feet in diamete external to station, and are constructed in reinforce concrete, cast-iron pipes not being available.

To make use of the syphonic action it was necessary t drown permanently the main circulating water pumps necessitating the sinking of a pumping station to belo the level of the lowest tide so that the pumps themselve are some 62 feet below finished ground level. The tur bine house basement is 22 feet below finished groun level, and the maximum economic syphonic assistanc obtainable is about 15 feet, the circulating water dis charge ducts being built immediately beneath the base ment floor.

River water intakes are common enough, but that a Uskmouth is somewhat exceptional since it is locate right at the mouth of a large tidal estuary. Fig. 1 shows the tidal ranges at spring and neap tides. Th pumping station was built as a caisson and sunk t calculated levels. Its main plant consists of eigh vertical spindle centrifugal pumps, each of 2,500,000 ga capacity and 18 band screens. The overall dimension o the caisson is 109 ft. 6 in. × 164 ft. 0 in. and this was sun to —51.75 with a finished top floor level of +28.75. Th superstructure rises to level 62·00.

The geological formation is also indicated. The firs operation was to drive a protective steel sheet piled wal as shown, and to prepare the compressor housing, base etc., required for the sinking. At "A" 350 tons o welded steelwork were assembled to form three workin chambers with safety tunnel connections.

Whilst this was proceeding on the shore side of th steel sheet piling, the contractors were excavating a leve area on the foreshore under "B". Here a 4 ft. thick bric rubble mat lay ready to receive the steel workin chamber when rolled out on ball-races. Its weight wa transferred to four pairs of steel tubes drive perpendicularly into the foreshore, but before lowerin to brick-rubble mat a convenient range of tides had to b awaited to make possible the concreting which was t sink the cutting edges to a calculated depth of —51.7 O.D., i.e. to "C". Letters "A", "B" and "C" clearl indicate the main stages.

Consideration had also to be given to the cill level o the dock entrance opposite, directly related to the main tained dredged level below it. To enable water to b taken from the river at all states of the tide, deepe dredging was necessary to form a channel that woul feed the pumps.

The natural slope of the river banks was one in eigh and steeper slopes therefore necessitated steel shee piling and fascined and armoured inclines.

Rolling Out

As originally designed, this pumping station wa divided into two identical units of 84 ft. 9 in. × 119 ft 6 in. with a 3 ft. 6 in. space for jointing after sinking This connection was to be made within 80 feet long stee sheet piling, and provision was made for "limpet" par piles to be incorporated in each unit. The cofferdam wa to be large enough to include the foundation for th rough screens which prevent damage to the installatio when the river in spate carries heavy debris.

In the actual work a similar cofferdam was used heavily framed in structural steelwork to counter th high tides, rising sometimes to +22.00 O.D.

Before letting this contract, boreholes were taken and a 5 ft. 0 in. diameter well sunk to enable the ground water levels to be compared with tide levels.

A complete report showing soil characteristics, sizing analysis, shear resistance and internal friction angle, moisture content, liquid limit and plastic limit was attached to the tendering documents. Two important facts were revealed :—

(a) The average bearing capacity of the subsoil would be in the region of .33 to .4 tons per square foot.

(b) The water level in the boreholes was generally far lower than the tide water level.

It was contended that a more or less constant water level would be found at approximately datum, or possibly still lower, yet the contractor had to be warned that this could develop at a higher level. Pressure on the ground might therefore well have to be kept within a limit of ¼ ton per square foot in the early stages of sinking, but increasing on reaching greater depths.

Messrs. Holloway Bros. Ltd., asked to be allowed to submit an additional tender based on using one in place of two units. The engineers agreed to this and adjusted their details accordingly. In this way the Uskmouth caisson became the largest of its type yet sunk. The general dimensions are as follows :—

Length	164 ft.
Breadth	109 ft. 6 in.
Toe (cutting edge)	—51.75 O.D.
Pump House Floor Level	+28.75 O.D.
Gantry Roof Level	+62.00 O.D.

The pump house superstructure is steel framed and clad in brick to match the main building. Two 15 ton gantries span the twin pump chambers for easy handling of the eight vertical spindle pumps and ancillary plant, and two 7½ ton gantries and the two screen chambers.

Prestressed Bridge over Sidings

Road access to the station consists of a level-crossing and an elevated roadway running parallel to each other across 18 siding tracks. The level-crossing is reserved for exceptionally heavy loads, such as transformers, whilst everyday traffic will use the elevated roadway, carried by a multi-span prestressed concrete bridge some 700 ft. long with access ramps at both ends.

This bridge has 13 intermediate supports consisting of plain reinforced concrete walls 2 ft. 6 in. thick and 14 ft. 6 in. broad, founded on cored piles. The piers are placed so as to give a minimum clearance of 5 ft. 6 in. to the existing rails and divide the bridge length into 11 spans of about 58 ft. and three spans of 30 to 35 ft.

The deck structure provides a 9 ft. roadway and a 2 ft. 6 in. footpath with service ducts. It consists of a prestressed concrete solid slab requiring the minimum structural depth for the loads envisaged and, therefore, improving the gradient at the access ramps. The slab structure is obtained by placing precast beams of a rectangular cross-section side-by-side and assembling them laterally by means of transverse post-stressing cables. This method of erection is particularly valuable as it hardly interferes at all with the rail traffic.

The precast beams, or slab strips, are 12 in. wide and 36 in. deep, containing two to four Freyssinet cables according to the span. A 1 in. gap left between adjacent units is packed tight with cement mortar before tensioning the transverse post-stressing cables to form a monolithic slab. Each span is provided with a reinforced concrete rocker-bearing at one end and a medium hard steel roller at the other.

The carriageway is surfaced with a layer of tarmacadam laid to falls between precast kerbstones.

Roads and Haulage

In large modern power stations the handling of awkward and heavy loads up to 175 tons calls for care in planning.

The road or rail haulier have to-day specially constructed vehicles to deal with such loads. It is therefore necessary for the civil engineer to consider the economics of the position. If the operation is such that a large awkward load has to be positioned once in, say, the life of the station, then the haulier can be paid to carry out special methods of handling as they are non-recurring—but if such loads may be moved several times during the life of the station, then the roadways and haulages should be such that they are permanently provided.

Generally speaking ferro-concrete roadways are preferable to tarmacadam.

In planning a station such matters must be considered early in the design.

Drainage and Sewage Disposal

Most of the seven acres of roof area of the main station are drained by leading the rainwater over screeding and through down pipes, into a channel running along the whole length of the lowest roof. The water thus collected enters the 24 in. × 12 in. vertical shafts inside the end facades and discharge into detritus pits which remove grit, etc. and allow the water to be passed over weirs into the storm drains. The remainder of the roof area is drained by concealed downpipes which are syphoned at ground level. The lesser buildings, as well as the roads in the immediate vicinity of the station, are treated in the more conventional manner.

The foul water system has its own disposal works to avoid polluting the river. Neither foul nor storm drains lie under the concrete roads themselves.

Amenities

Canteens, recreation rooms, libraries, first-aid, ablution and lavatory facilities all play an important part in power station design to-day, and are planned most carefully to anticipate staff requirements.

Weigh houses, a control lodge, fencing and the multitude of small works which go to complete a modern power station have, of course, also been installed.

Fig. 13 shows the station nearing completion. The 360,000 kW which the Uskmouth Station will contribute to the national grid system can be easily appreciated as an important increment in power production. Its ultimate value in the historical progression whose beginnings have been outlined in this paper is, however, not so readily seen. But what is clear is that, although it incorporates no revolutionary new processes, those of its modest innovations which were before but academic speculation have now become established fact, and another page of a chapter has been written.

The Building and Civil Engineering Contracts were divided into four main groups as follows :—

(1) Main Power Station, Switch-House, Roads, Bridges, Circulating Conduits, etc.—Messrs. John Morgan (Builders) Ltd., Cardiff.

(2) Railway Sidings—The Isca Foundry Co., Ltd., Newport.

(3) Structural Steelwork—Messrs. Braithwaite & Co., West Bromwich.

(4) River Water Intake and Ancillary Works—Messrs. Holloway Bros. (London) Ltd.

The Consulting Civil Engineers for the whole of the Civil and Building Works are Messrs. L. G. Mouchel & Partners Ltd., acting in harmony with the British Electricity Authority's Chief Engineer Mr. V. A. Pask, M.I.E.E., M.I.Mech.E.

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The Interplay of Concrete of Different Ages in Structural Members

By Professor Dr. Ing. H. Craemer

Summary

The distribution of stresses in old and new concrete in compound sections is examined for an instantaneous load, after which the redistribution of stresses under sustained load and shrinkage is described. The redistribution of stress which takes place in statically indeterminate structures when their individual members are cast independently and connected later is discussed.

Effect of Strengthening of a Concrete Member

If a reinforced concrete section has to be strengthened because the working loads have been increased above those originally provided for, the original and the new parts are stressed in different ways. Even in the simple case of a centrally loaded strut strengthened by a concertriasheath, Fig. 1, the stresses are different in the two parts. The part of the total load which existed before the strengthening was finished will affect only the inner, older, part of the strut while the loads applied later will stress both old and new concrete. In order to obtain as even a stress distribution as possible, the structural member in question is usually relieved as much as possible of its load during the strengthening process. The stress distribution in the two parts of the cross-section can be determined without difficulty by applying the elastic theory.

Now, the strains which influence the stress distribution increase in the course of time if the load is sustained. This *time yield*, or creep, is greater in new than in old concrete, with the result that there is a redistribution of stress over the cross-section causing the stresses in its two parts to become more nearly equal. Thus part of the load acting before strengthening is transferred from

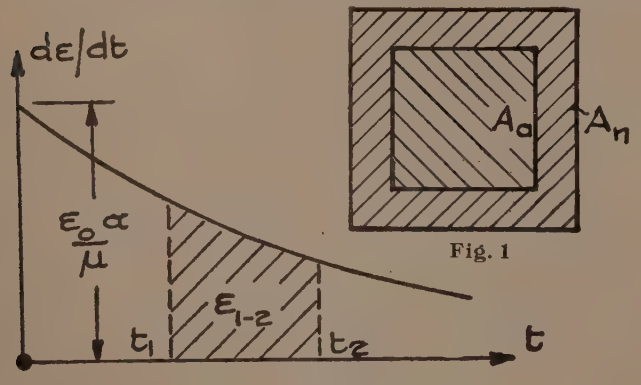


Fig. 2

Fig. 1

the older part to the newer. At the same time a load applied after strengthening, and therefore acting on both parts, tends to transfer to the older, more rigid, concrete. These phenomena can be investigated theoretically by generalising the stress-strain-law by including the variability with time.

Shrinkage, too, is different in the two parts of a strengthened section, as old concrete shrinks less than new. By its greater shrinkage the new part induces compressive stresses in the older material which increase in the course of time, whereas the compressive stresses in the new part due to the external applied load tend to lessen. The laws governing this phenomenon are similar to those concerned with creep under sustained load.

Until now we have referred to working loads, and therefore to stresses which are well below the failure point. In the case of an overload, however, the linearity of the stress-strain-law is no longer valid for either instantaneous or sustained stresses. In the overstressed parts, the strains increase more rapidly than the stresses and, as they transmit these deformations to the less affected parts connected with them, the load is transferred from the highly stressed parts to the others and a further redistribution takes place.

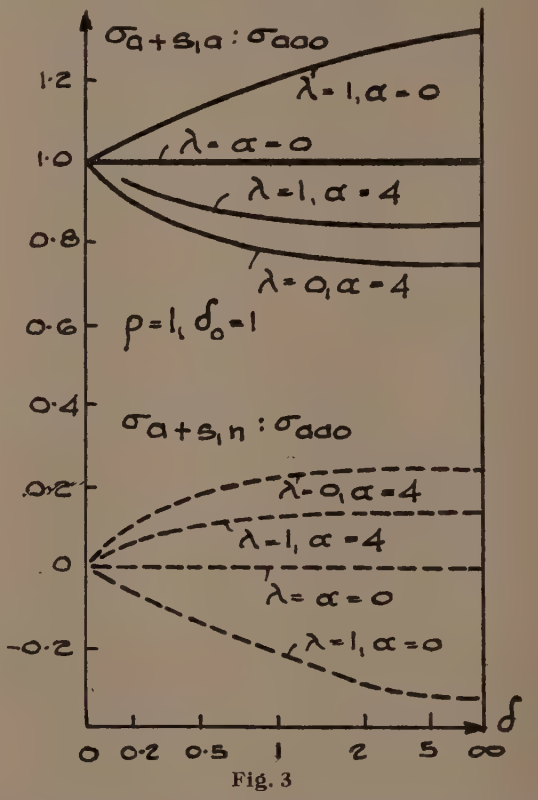


Fig. 3

What has been already discussed applies in a similar way to any extension of a statically indeterminate structure, say, when some more bays are connected with an already finished and loaded, continuous beam. In such a case the stresses are at first different from those ocurring when the entire system has been erected as a whole and then loaded; a balance, however, takes place in the course of time so that at the end, the different ages of the two parts have no longer an important influence.

The same ideas hold when a cross-section is assembled out of individual parts poured at different times, as, for instance, a vault consisting of several rings situated above or beside one another. In the following the effects described will be examined more elaborately.

Stresses Immediately after Strengthening

Suppose a cross-section, A_a, is strengthened by a new part, A_n, with a common centroid and let a normal force, N, act concentrically. If the load before strengthening is N_{aa}, it produces a stress

$$(1)\quad \sigma_{aa} = \frac{N_{aa}}{A_a}.$$

In the following discussion the first suffix always denotes the cause (a = old load, n = new load, s = shrinkage),

the second one the place of action (a = old, n = new part of cross-section). Strictly speaking the stated stresses are fictitious as the stress in the reinforcing rods is m times that in the concrete, where m is the modular ratio.

Let the new part of the cross section be

$$(2)\quad A_n = \rho A_a.$$

The load applied after the strengthening acts on both parts and will therefore be denoted by $N_{n,\,a+n}$; it produces the same stress in both parts thus

$$(3)\quad \sigma_{na} = \sigma_{nn} = \frac{N_{n,\,a+n}}{(1+\rho)\,A_a} = \sigma_{n,\,a+n}.$$

A possible difference in the elastic moduli in old and new concrete has been neglected but this however, could easily be taken into account.

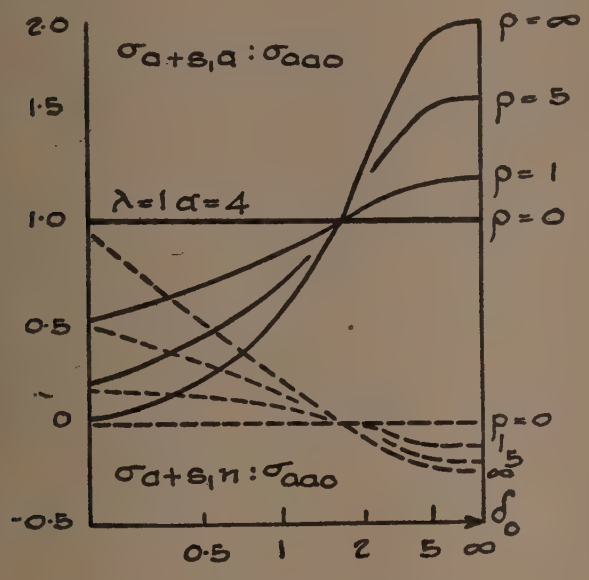


Fig. 4

We now refer the new load to the original one by

$$(4)\quad N_{n,\,a+n} = \nu\, N_{aa}.$$

Without the strengthening, the total load, $(1+\nu)\,N_{aa}$, would induce a stress of

$$\sigma^1{}_a = \frac{(1+\nu)\,N_{aa}}{A_a}$$

in the unstrengthened part A_a. After strengthening, however, we get from equations (1), (3) and (4)

$$\sigma_{a+n,\,a} = \left(1+\frac{\nu}{1+\rho}\right)\frac{N_{aa}}{A_a}.$$

Now, the efficiency of the strengthening can be co-ordinated to a value $\gamma = \sigma_{a+n,\,a}/\sigma^1{}_a$, thus

$$(5)\quad \gamma = \frac{1+\rho+\nu}{1+\rho+\nu+\rho\nu}.$$

Without any strengthening, $\rho = o$, we get $\gamma = 1$; the same, however, is obtained from $\nu = o$, *i.e.* by applying the whole load before the new part is finished; thus, at present, the enlargement of the cross-section seems to be of no use. On the other hand, when the section is completely relieved during construction and the whole load applied after this, by $\nu = \infty$ we get $\gamma = 1/(1+\rho)$.

The same relations hold good for the action of bending moments if both parts of the cross-section have the same centroid; ρ in this case is defined as the ratio of the moments of inertia, $\rho = I_n/I_a$. The stresses at a distance y from the common neutral axis using symbols corresponding to the previous ones are then

$$(6)\quad \sigma_{aa} = \frac{M_{aa}\,y}{I_a} \text{ and}$$

$$(7)\quad \sigma_{ba} = \sigma_{nn} = \frac{M_{n,\,a+n}\,y}{(1+\rho)\,I_a}.$$

Stress Transfer in the Elastic State

(a) *Dependence of time-yield on time.*

Let the instantaneous strain be

$$(8)\quad \varepsilon_o = \frac{\sigma}{E}.$$

In connection with the influence of time-yield on the stress redistribution, several authors have contended that its end value depends only on the end value of the time-yield function but not on the intermediate values of the latter in the course of time. I have pointed out in a previous paper[1] that this is not correct. We therefore have to assume an analytical expression for the dependence of creep on time or, instead, for the creep velocity, *i.e.* the increase of creep per unit time. We assume, see Fig. 2,

$$(9)\quad \frac{d\varepsilon}{dt} = \frac{\alpha}{u}\,\varepsilon_o - e^{-t/u}$$

or, to make the arithmetic easier

$$(10)\quad \frac{d\varepsilon}{d\delta} = \varepsilon_o\,\alpha\,e^{-\delta},\quad \delta = t/u.$$

In this expression t is the *age* of the concrete (not the duration of loading). Thus, with increase in t, the creep velocity decreases; so the equation expresses the experimental fact that new concrete is more liable to creep than old; at a time $t = \infty$ there is no further deformation. Since immediately after pouring, *i.e.* when $t = o$ the material has not developed its full strength, the moment of sufficient hardening should be defined.

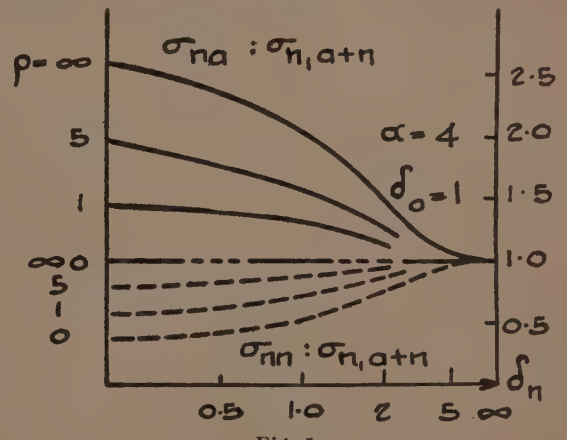


Fig. 5

The time-yield produced within a period $t = t_1$ to $t = t_2$ is obtained by integration $\varepsilon_{1\text{-}2} = \varepsilon_o\,\alpha\,(e^{-\delta_1} - e^{-\delta_2})$; it corresponds to the shaded area in the figure; δ_1 and δ_2 are defined by equation (9). When $t_1 = o$ and $t_2 = \infty$ which corresponds to an infinitely long loading, we get

$$(11)\quad \varepsilon_{o-\infty} = \alpha\,\varepsilon_o.$$

Therefore, α is the rate of increase of the instantaneous strain in the worst case.

The term u has the dimension of time. Since, at $t = o$, $\frac{d\varepsilon}{dt} = \frac{\varepsilon_o\alpha}{u}$ (see Fig. 2), u is inversely proportional to the initial velocity when the load is applied just after hardening; the final values of the deformation are not influenced by u. By an appropriate choice of u, equation (9) can be made to agree with experimental results. The author has found that roughly

$$(12)\quad u = 100\ \textit{days}.$$

A fuller discussion of the reasons leading to the assumption of equation (9) is given in a previous paper by the author.

(b) *Stresses due to a load applied before strengthening and sustained after it.*

Again we examine a concentrically strengthened strut under the action of a normal load. Let $t_o = \delta_o u$ be the difference in age of the two parts of the section, t the age of the newer concrete; hence $t + t_o$ is the age of the older concrete. After strengthening, the deformation of the older part will increase by an amount $d\varepsilon_{aa} = \frac{d\sigma_{aa}}{E} + \frac{\sigma_{aa}}{Eu} \alpha e^{-(\delta + \delta_o)}\, dt$ during an interval dt; the first term is the instantaneous strain due to the change of stress, $d\sigma_{aa}$, the second is the time-yield according to equation (10). Instead we can write

(13) $Ed\varepsilon_{aa} = d\sigma_{aa} + \alpha\sigma_{aa} e^{-(\delta + \delta_o)}\, d\delta$.

Correspondingly for the new part

(14) $Ed\varepsilon_{an} = d\sigma_{an} + \alpha\sigma_{an} e^{-\delta} d\delta$ is obtained.

Now, if the normal force is kept constant during the redistribution, we have

(15) $d\sigma_{aa} + \rho d\sigma_{an} = o$

or, by integration

(16) $\sigma_{aa} + \rho\sigma_{an} = \sigma_{aao}$,

σ_{aao} being the stress in the original part at $t = o$ since at that moment no load has been transferred to the new part, *i.e.* $\sigma_{an} = o$ and $\sigma_{aa} = \sigma_{aao}$.

Now, provided the bond is perfect, $d\varepsilon_{an} = d\varepsilon_{aa}$, *i.e.* the two parts can deform only together. By taking this into account in equations (13) and (14) and inserting equation (16), we get a differential equation in σ_{aa} and δ; the mathematical work involved has been described earlier.[2]

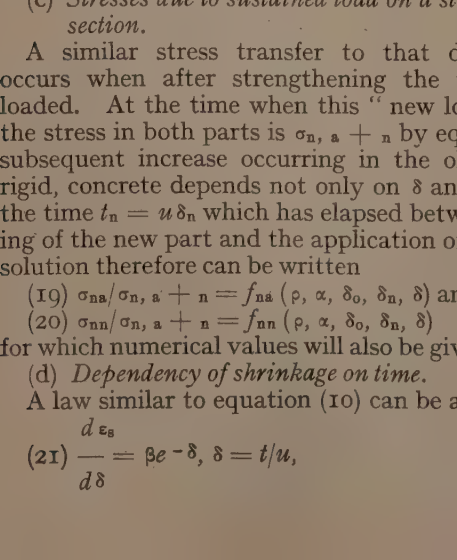


Fig. 6.

The result can be expressed in the form

(17) $\sigma_{aa}/\sigma_{aao} = f_{aa}(\rho, \alpha, \delta_o, \delta)$ and

(18) $\sigma_{an}/\sigma_{aao} = f_{an}(\rho, \alpha, \delta_o, \delta)$.

A numerical evaluation will be given later on. The same relations hold good for bending stresses and a strengthening by $\rho = I_n/I_a$ with the same centre.

(c) *Stresses due to sustained load on a strengthened cross-section.*

A similar stress transfer to that described above occurs when after strengthening the total section is loaded. At the time when this " new load " is applied, the stress in both parts is $\sigma_{n,a} + {}_n$ by equation (3); the subsequent increase occurring in the older, thus more rigid, concrete depends not only on δ and δ_o but also on the time $t_n = u\delta_n$ which has elapsed between the hardening of the new part and the application of the load. The solution therefore can be written

(19) $\sigma_{na}/\sigma_{n,a} + {}_n = f_{na}(\rho, \alpha, \delta_o, \delta_n, \delta)$ and

(20) $\sigma_{nn}/\sigma_{n,a} + {}_n = f_{nn}(\rho, \alpha, \delta_o, \delta_n, \delta)$

for which numerical values will also be given.

(d) *Dependency of shrinkage on time.*

A law similar to equation (10) can be assumed :

(21) $\frac{d\varepsilon_s}{d\delta} = \beta e^{-\delta}$, $\delta = t/u$,

ε_s being the shortening per unit length after a period t the case of unrestrained shrinkage. The time u has t same meaning as discussed in paragraph (a); lat we shall, for convenience, assume that the u's for cre and shrinkage are the same. By integrating betwe the limits $o \leqq \delta \leqq \infty$ we get $\varepsilon_s = \beta$, β thus being t strain due to free shrinkage after an infinite time. W further introduce a term (22) $\sigma_s = E\beta$ representing t tensile stress in a concrete member if its ends were r strained from moving and if there was no relaxation d to creep.

(e) *Stresses due to shrinkage in a compound cross-sectio*

The contraction of the original part due to shrinka and the stresses induced thereby increases during period dt by

$$d\varepsilon_{sa} = \frac{d\sigma_{sa}}{E} + \left(\alpha\frac{\sigma_{sa}}{E} + \beta\right) e^{-(\delta + \delta_o)} d\delta,$$

the first term on the right being the instantaneous stra resulting from the change in stress, $d\sigma_{sa}$, the second is t creep due to σ_{sa}, and the third the shrinkage itse Instead we can write

(23) $Ed\varepsilon_{sa} = d\sigma_{sa} + (\alpha\sigma_{sa} + \sigma_s) e^{-(\delta + \delta_o)} d\delta$

and correspondingly for the new part,

(24) $Ed\varepsilon_{an} = d\sigma_{sn} + (\alpha\sigma_{sn} + \sigma_s) e^{-\delta} d\delta$.

Similarly to equation (15) we have $d\sigma_{sa} + \rho d\sigma_{sn} =$ As $d\varepsilon_{sa} = d\varepsilon_{sn}$, we can get, by combining the abo equation with equations (23) and (24), a differenti equation for either σ_{sa} or σ_{sn}. The solutions can be presented the form

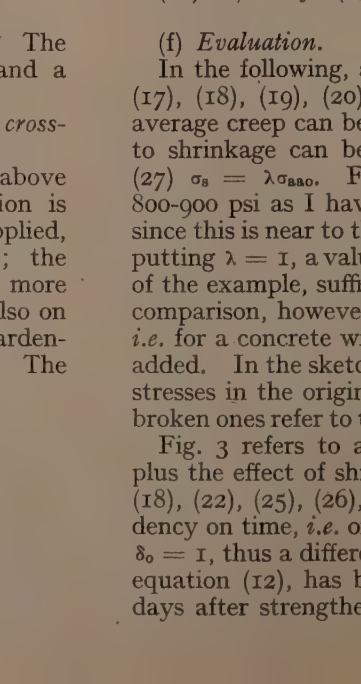


Fig. 7.

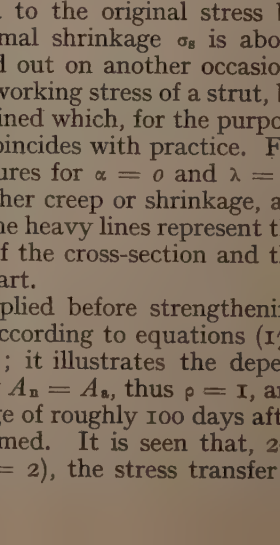


Fig. 8.

(25) $\sigma_{sa}/\sigma_s = f_{sa}(\rho, \alpha, \delta_o, \delta)$ and

(26) $\sigma_{sn}/\sigma_s = f_{sn}(\rho, \alpha, \delta_o, \delta)$.

(f) *Evaluation.*

In the following, a numerical evaluation of equatio (17), (18), (19), (20), (25) and (26) will be given. average creep can be estimated by $\alpha = 4$. Stresses d to shrinkage can be referred to the original stress (27) $\sigma_s = \lambda\sigma_{aao}$. For a normal shrinkage σ_s is abo 800-900 psi as I have pointed out on another occasio since this is near to the usual working stress of a strut, putting $\lambda = 1$, a value is obtained which, for the purpo of the example, sufficiently coincides with practice. F comparison, however, the figures for $\alpha = o$ and $\lambda =$ *i.e.* for a concrete without either creep or shrinkage, a added. In the sketches, all the heavy lines represent t stresses in the original part of the cross-section and t broken ones refer to the new part.

Fig. 3 refers to a load applied before strengtheni plus the effect of shrinkage according to equations (1 (18), (22), (25), (26), and (27); it illustrates the depe dency on time, *i.e.* on δ. Now $A_n = A_a$, thus $\rho = 1$, a $\delta_o = 1$, thus a difference in age of roughly 100 days aft equation (12), has been assumed. It is seen that, 2 days after strengthening ($\delta = 2$), the stress transfer

nearly complete. It is further seen that, in the case of a concrete not subject to creep ($\lambda = 1$, $\alpha = 0$) the older part is compressed more and more while the new part is stressed in tension; thus the strengthening would be completely useless in the working state (not, however, at failure). If the yield with time is taken into account ($\alpha = 4$), the stress decreases in the old concrete but increases in the new; the old concrete, however, still carries a stress of three times that in the new even if there is no shrinkage; the balance, therefore, is far from being complete.

In the following figures only the end values are stated ($t = \delta = \infty$). Fig. 4, again, refers to a sustained load on the original section plus shrinkage, the influence of ρ and of the difference in age, expressed by δ_0, being examined on the assumption $\alpha = 4$, $\lambda = 1$. If the difference in age is small, $\delta_0 < 1.6$ the redistribution is the more distinct the greater the rate of strengthening, ρ. Beyond that, $\delta_0 > 1.6$, thus, with differences in age of more than 160 days the harmful influence of shrinkage prevails and the older concrete is further stressed instead of being relieved, the effect becoming greater with increase in the amount of new concrete added to the section.

The end values of a stress redistribution due to a load applied after strengthening are plotted in Fig. 5, the differences in age being assumed as $\delta_0 = 1$. According to equations (19) and (20) the time $t_n = u\delta_n$ between strengthening and loading is involved. If 500 days are waited ($\delta_n = 5$), a redistribution no longer takes place and a sustained load behaves like an instantaneous one. If a load is applied early as in practice, however, an appreciable part of it is carried by the older material.

Redistribution of Stresses due to Overload

Concrete does not obey Hooke's law, its strain-stress-relation has nearly the shape shown in Fig. 6. In order to make the mathematical work easier and the interpretation of the phenomena clearer, a purely elastic and a purely plastic range of deformation may be assumed according to the inclined and horizontal broken lines. The plastic range can be limited by ε_p = about 0.001; if the lateral expansion of the piece is prevented, say by spiral reinforcement, a multiple of this value is reached.

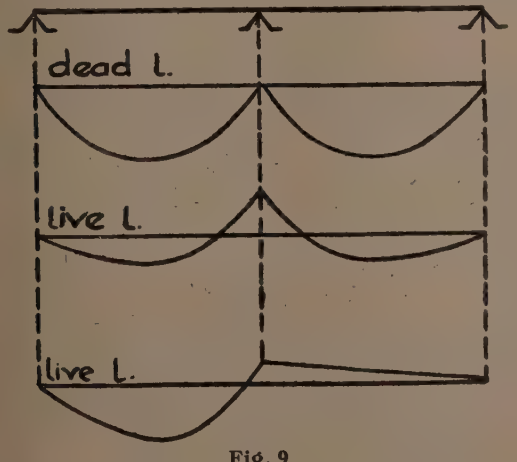


Fig. 9

Now, assume a strut to have certain stresses σ_a in the old part, and σ_n in the new, due to the original load, new load and shrinkage. Let an additional load, ΔN, be applied and increased until failure occurs. Since σ_a is always $> \sigma_n$, the old material first reaches its breaking stress. This, however, does not yet mean that the strut itself fails, since now the old material deforms plastically while the new still remains elastic, see Fig. 7.

If now, the distance between the two deformation lines is not greater than ε_p, the new concrete will overtake the older and, for its part, reach the breaking stress. It is not before this point is reached that a further increase of loading is impossible, the strengthened column thus has the same ultimate bearing capacity as if the whole cross-section had been constructed at the same time.

Thus, as long as $\varepsilon_p \geqq \frac{\sigma_a - \sigma_n}{E}$, the bearing capacity at failure can be computed regardless of the initially different stresses in the two parts, see Fig. 7. Under the very unfavourable assumption $\sigma_a - \sigma_n = 1{,}500$ psi and $E = 3{,}000{,}000$ psi, $\varepsilon_p = 0.0005$ is obtained; this value is

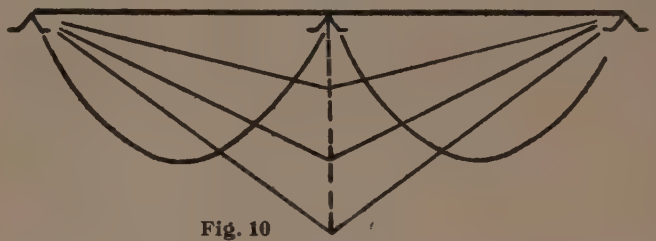


Fig. 10

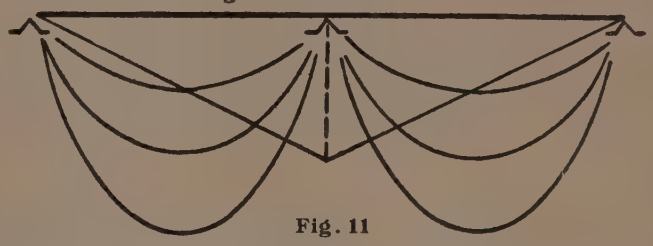


Fig. 11

reached in any case. In the region of failure, therefore, the behaviour of a strut is *independent* of the difference in age of the parts making up its cross-section and of the sequence, magnitude and duration of the individual loadings.

Moreover, Fig. 8 illustrates that the gradually increasing stresses are dependent on the increases of load, ΔN; as long as both parts of the cross-section remain in an elastic state, the lines σ_a and σ_n are parallel. After σ_a has become equal to the failure strength, σ_n will increase more rapidly until the new part is also stressed to failure and thus the ultimate load is reached. Since, in reality, the stress-strain line in Fig. 6 is curved, the corners in Figs. 7 and 8, strictly speaking, have to be rounded off.

Construction of Statically Indeterminate Systems by Segments

Statically indeterminate systems, especially continuous beams, are frequently erected in two or more segments which are connected after removal of the formwork; this procedure is particularly favoured for two-span hangars and bridges. If in this way the joint over the middle support is closed after removal of the formwork of both bays, there will be no bending moment at the support due to self weight. Since this bending moment often causes difficulties in design, it is hoped in this way to obtain a valuable reserve for carrying the subsequently applied live load, Fig. 9.

Now, the deformations on which the magnitude of the redundant forces in statically indeterminate structures depend, occur only partially at the instant the load is applied, *i.e.* on removal of the formwork. If at this time the concrete is comparatively fresh, the main deformation will take place some considerable time later. If, therefore, the joint is connected soon after removal of the formwork, the greatest part of the deformation will take place in the connected system. A more elaborate investigation[2] shows that the moment at the support, which it was intended to remove, will rise to almost its full value after some time, the closing line thereby being changed in the manner shown in Fig. 10.

Thus, in order to remove this negative moment completely, it would be necessary to wait until the concrete ceases to be liable to creep, that is to say, almost a year. A similar reasoning holds when the formwork is removed from one span first, then the joint connected and later the formwork removed from the second span. If the age of the two spans is different, the behaviour can be examined in a similar way.

The concrete tension members of reinforced concrete trusses is frequently poured after the supports have been removed from the truss. If it is a roof, the load on which consists mainly of its own weight, the greater part of the tensile force thus acts before the concrete of that member is poured. The designer thus intends not only to avoid cracks but also to eliminate the bending moments which occur especially in concrete trusses as a result of the stiff joints and the relatively great moments of inertia. It is true that, as the reinforcing rods have no stiffness, there are no bending moments at the joints just after pouring. Since, however, the deformations continue, subsequently induced moments are almost as great as if the whole structure had been concreted before removal of the supports. Under normal conditions, therefore, these methods gain only a small part of the advantages with which they are credited as a result of insufficient consideration of the time-yield.

This behaviour changes near the failure point. The redundant forces in indeterminate systems mostly differ materially from those computed by the elastic theory. Suppose, for instance, the beam of Fig. 9 is designed in such a way that after the stress redistribution due to creep and shrinkage is completed, the stress in the reinforcement over the support is greater than that in the other sections; in the event of an overload the yield stress will be reached first at this place. If the load is raised further, the moment at the support can no longer increase and a so-called plastic hinge will form itself at that section. For equilibrium the moments in the spans must increase more rapidly than the loads, that is, the bending moment diagram will change as shown in Fig. 11. Finally, the reinforcement in the span, too, is stressed to the yield point and it is not before this that a further increase of the load becomes impossible. Thus, at failure, a further redistribution of stresses results, the final values of which are *independent* of the foregoing redistributions due to creep and shrinkage.

References

[1]H. Craemer. Beton und Eisen 43 (1944) No. 1 - 2. (In German.)

[2]H. Craemer. Ingenieurarchiv 14 (1943) pp. 119 - 135. (In German.)

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1. on Thursday, November 27th, 1952, at 5.55 p.m. Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President) in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

ABRAHAMS, Frank Dudley, of Birmingham.
AGER, John Stanley, of London.
ASTILL, Alan Walter, of Nottingham.
BARTLETT, Harold Alfred, of Salisbury, Southern Rhodesia.
BROWN, Donald Robert, of Southland, New Zealand.
BROWN, Geoffrey Hewitt, of Winsford, Cheshire.
BURKE, Jeremy Ulick, of London.
CLUFF, Alfred William George, of London.
COWAN, John, B.Sc.(Civil)Edinburgh, of Edinburgh.
DAINES, Alfred Lionel, of Chatteris, Cambridgeshire.
DAWSON, James Alfred Jack, of Erith, Kent.
DEAR, Anthony John, of Southampton.
DODD, Harry Thomas, of Birmingham.
GARLICK, Frederick Hugo, of Newquay, Cornwall.
JONES, Emrys, of Port Talbot, Glam.
KIRBY, Norman, of Gateshead 9. Co. Durham.
KNOTT, Stanley William, of Manchester.
LANG, David Christopher Beswick, of Germiston, S. Africa.
LANGHAM, Bernard Trevor, of Isleworth, Middlesex.
LEE TING-WAI, B.Sc.(Eng.)HongKong, of London.
MEHTA, Kedar Nath, of Drighlington, Nr. Bradford.
MEHTA, Navinchandra Raghunath, of Nairobi, Kenya.
MILLMAN, James Alan, of Wellington, Shropshire.
MULHALL, Kenneth, of Warrington, Lancs.
PRIOR, Anthony Paul, of London.
ROGERS, Kenneth Joseph Charles, of Wellington, New Zealand.
SEEDHOUSE, Peter Duffell, of Birmingham.
STANLEY, Brian William, of Stockton Heath, Nr. Warrington, Lancs.
TAYLOR, Donald Owen, of Port Talbot, Glam.
TAYLOR, Geoffrey Charles, of Warrington, Lancs.
THOMAS, Martyn Robin Haden, of Birmingham.
THOMPSON, Frank, of Manchester.
WILLCOX, Kenneth Leslie, of Peasedown St. John, Nr. Bath, Somerset.
WOODHOUSE, James, of Allestree, Derby.

GRADUATES

ABBS, Ronald Samuel, of Cadishead, Nr. Manchester.
ALLINGTON, Reginald Arthur, of London.
ANGUS, John Martin, of London.
ARCHIBALD, Allan Edwin, B.Sc.(Eng.)Rand, of Johannesburg, S. Africa.
ATKINSON, Frank Brian, of Johannesburg, S. Africa.
BALLARD, Eric Hugh, B.Sc.(Eng.)London, of Enfield, Middlesex.
BARREY, Alan Erskine, B.Sc.(Civil)Durham, of Sunderland.
BURGESS, William Fargie, of Inverkeithing, Fife, Scotland.
CAREY, Dudley Layton, of Taranaki, New Zealand.
CHENHALL, Raymond Leslie, of Stapleton, Bristol.
COLLIS, John Watts, of London.
COY, Sydney David, of Denton, Lancs.
DAVID, Solomon Arulanandam, of Colombo, Ceylon.
DHARWARKAR, Prabhakar Dhondopant, of Belgaum, Bombay State, India.
DU TOIT, George Alfred, of Cape Town, South Africa.
DU TOIT, Pieter Schalk, of Johannesburg, South Africa.
EFFIONG, Maurice Okon, of Lagos, Nigeria.
FOTHERINGHAM, John Henderson, of Johannesburg, South Africa.
FRANCIS, Herbert Bramwell, of Napier, New Zealand.
FREER, Charles Noel, of Leicester.

GIBSON, John Hartt, B.Sc.(Eng.)London,of Birmingham.
GOH JOON YAP, of Fort Canning, Singapore.
GREEN, Edward Ellis, of Norwich, Norfolk.
GREEN, Reginald William, of Manchester.
HAINES, Warwick Clive, of Cardiff.
HARRISON, Norman, of Chesterfield, Derbyshire.
HAWORTH, Brian Singleton, B.Sc.(Rand), of Salisbury, Southern Rhodesia.
HODGE, Basil Kenneth, of Oakengates, Shropshire.
HORRIDGE, James Frederick, of Bolton, Lancs.
HORTON, William John, B.Sc.(Civil) Cape Town, of Salisbury, Southern Rhodesia.
HUGGINS, Raymond Arthur, B.Sc.(Eng.) London, of London.
JACKSON, Anthony Derek Haygarth, B.A.(Hons.) Cantab. of Altrincham, Cheshire.
JAFFE, John Walter Paul, M.A. Cantab, of London.
JOSHI, Aravinda Vithal, B.E.(Civil) Poona, of London.
KHAN, Mahmood Aslam, of Rawalpindi, Punjab, Pakistan.
KINKAR, Laxmikant Neelkanth, B.E.(Civil) Poona, of Poona, India.
LAKIN, Noel Oscar Ernest, B.E.(Civil) Calcutta, of Newcastle upon Tyne.
LINSDALL, Kenneth Charles, of London.
LIVESEY, Keith Haydn, M.Sc.(Civil) Leeds, of Bo'ness, West Lothian.
MCCAFFREY, Dermot William, B.A., B.A.I., Dublin, of Suva, Fiji.
MAIRS, James Girvan, B.Sc.(Civil) Glasgow, of Glasgow.
MA SHUN-FAI, of Peterborough, Ontario, Canada.
MEINEKE, Cyril Christopher, of Johannesburg, South Africa.
MORGAN, William, of Sunderland, Co. Durham.
MORROW, James Robert, B.E.(Civil) New Zealand, of Canterbury, New Zealand.
MOSS, John Didsbury, B.E.(Civil) New Zealand, of Wellington New Zealand.
MUNNS, Thomas Trevor, of East Kirkby, Notts.
MUSGRAVE, Denis, of Scunthorpe, Lincs.
NICHOLS, John Russell, B.Sc.(Eng.) Cape Town, of Germiston, South Africa.
OGILVIE, Gavin, B.E.(Civil) New Zealand, of Christchurch, New Zealand.
PAICE, Charles Thomas, B.Sc.(Eng.) South Africa, of Transvaal, South Africa.
PATEL, Ramchandra Becharbhai, B.E.(Civil) Bombay, of Poona, India.
PHILLIPS, John Harry, A.M.I.C.E. of Hemel Hempstead, Herts.
PRETE, Peter Anthony, of London.
SALAM, Abdus, of Sargodha, Punjab, W. Pakistan.
SHINKWIN, Thomas Percival, B.E., N.U.I. of Dublin, Eire.
SHOTTON, James Edward, of Newcastle upon Tyne.
SINGH, Jattan, of Gorakhpur, India.
SULLIVAN, Eric Harold, of Bulawayo, Southern Rhodesia.
SUTHERLAND, Alexander John, of Greymouth, West Coast, New Zealand.
TALEB, Jameel Nazih, of Princeton, New Jersey, U.S.A.
TOPPS, Frederick Donald, of Middlesbrough, Yorks.
WALKDEN, Derek Melvin, of Brooklands, Sale, Cheshire.
WALKER, Geoffrey, of Messingham, Nr. Scunthorpe, Lincs.
WALMSLEY, John, of Bolton, Lancs.
WESTBURY, George Roy, of Sandiacre, Notts.
WILSON, James Sturrock, B.Sc.(Eng.) London, of Fao, Iraq.

ASSOCIATE-MEMBERS

AU SIK LING, of Hong Kong.
CHEN SHAN-CHUANG, of Hong Kong.
DREW, Ryszard Bronislaw, of Enugu, British West Africa.
EISINGER, Gerhard, of Jerusalem, Israel.
HANNA, Louis Demian, B.Sc.(Civil) Egypt, of Cairo, Egypt.
HILL, John Alexander, of Turramurra, New South Wales, Australia.
JOHANSEN, Cecil Clifton, of Plimmerton, Nr. Wellington, New Zealand.
LAING, Gordon Smith, B.Sc.(Civil) Aberdeen, A.M.I.C.E. of Johannesburg, South Africa.
LIAO CHUNG CHOU, Ph.D. Leeds, of Hong Kong.
LOWE, Robert William, D.F.C., A.M.I.C.E. of Nairobi, Kenya.
NAMAN, Ihsan, of Basrah, Iraq.
PARKYN, John Roy, of Nairobi, Kenya.
PATWA, Sohrab Navalshaw, B.Sc., B.E. Bombay, of Calcutta, India.
SHIROMANI, Shriman Rai, of Bombay, India.
STANDER, Ruben, B.Sc.(Civil) Cape Town, of Elsies River, Cape Province, South Africa.
WHITFORD, Eric Thomas, A.M.I.C.E. of Auckland C.1., New Zealand.

MEMBERS

CASSEL, Frederick Leon, A.M.I.C.E. of London.
HAYNES, Charles Duncan, of Bognor Regis, Sussex.

TRANSFERS

Students to Graduates

ALTRIA, Stephen Arthur, of London.
BAILEY, Edwin Roy, of Guildford, Surrey.
BOSMAN, Edward Albert, of Durban, South Africa.
BOTHMA, Kenneth Harold, of Johannesburg, South Africa.
BURR, Robert William, of London.
CALVER, Michael John, of Ipswich, Suffolk.
COLE, Brian Michael, of Birmingham.
DUNN, Allan George, of Coulsdon, Surrey.
HALSTEAD, Donald Harris, of London.
HIBBERT, Norman Trevor, of Urmston, Nr. Manchester.
HOUGHTON, Tom Keith, of Stockport, Cheshire.
JONES, Peter David Ellesmere, B.Sc.(Eng.) London, of Swansea, Glam.
LONGRIGG, Thomas Derek, of Nairobi, Kenya.
MORCOM, Peter John, of Durban, South Africa.
MORLING, Derek John Mariner, of Beckenham, Kent.
PARSONS, Dudley John Francis, of Carshalton, Surrey.
ROBSON, Ian Dudley, of Ruislip, Middlesex.
ROSE, Douglas Frederick, of London.
SAMPSON, Derick Stanley Maxwell, of Uxbridge, Middlesex.
TOBUTT, David Cooper, of Meopham, Kent.
TURTON, Kenneth, of Johannesburg, South Africa.
TWADDLE, Ian Edward, of Germiston, South Africa.

Graduates to Associate Members

BHANDARKAR, Prafullachandra Ganesh, B.Sc., B.E., Bombay, of Bombay 14. India.
BUCHAN, Alan James, of Wellington, New Zealand.
BURTON, Robert Bentley, of Wellington, New Zealand.
BYRNE, Bernard John Peter Le Cesne, B.E.,NUI. of Barnsley, Yorks.
CROSIER, Thomas Arnold, B.Sc.(Eng.) London, of Winnipeg, Canada.
CRUTCHLEY, Leslie, of Shirley, Nr. Birmingham.
DAVIS, Elsbury John, B.Sc.(Eng.) South Africa, of Durban, South Africa.
EYRES, Derek, of Northwich, Cheshire.
FROST, Lewis Lake, of Wallington, Surrey.
GRAFF, Jacob Yehuda, B.Sc.(Eng.) Rand, of Johannesburg, South Africa.
GROYER, Edward, B.Sc.(Eng.) Cape Town, of Johannesburg, South Africa.

HARRISON, Edward, of Hatton, Derbyshire.
LEE, Gerald Stanley, of Prestwich, Manchester.
LIGHTBAND, Michael Stuart, B.Sc.(Eng.) London, A.M.I.C.E., of Karachi, Pakistan.
MAILLARDET, Roy, of London.
MANN, Frank Thomas, B.Sc.(Eng.) London, of Hastings, Sussex.
ORME, Donald Harrison, M.A. Cantab, of Birmingham.
RUDD, Arthur Claude, of Dunedin S.W.1. New Zealand.
SCHAFFNER, Robert Hastings, B.Sc.(Civil) Rand, of Johannesburg, South Africa.
SHAH, Navinchandra Ambalal, B.E.(Civil) Bombay, of Baroda, India.
SHVARTZ, David Itzhak, of Tel-Aviv, Israel.
STOCH, Solly Gerald, B.Sc.(Eng.) Rand, of Johannesburg, South Africa.
WARNER, John Spearman, B.Sc.(Eng.) South Africa, of Johannesburg, South Africa.
WYATT, Christopher Terrel, B.Sc.(Eng.) London, of Clevedon, Somerset.

Associate Members to Members

BUNCLARK, Cyril Henry, B.Sc. London, A.M.I.C.E., of Purley, Surrey.
MATHEWSON, Kenneth Douglas, B.A.(Hons.) Cantab, M.I.C.E., of Singapore.
NEW, Dudley Holt, B.Sc.(Eng.) London, M.I.C.E., A.M.I.Mech.E., of London.

RE-ADMISSIONS

Associate-Members

FIRTH, Harry, of London.
SUBRAMANIAN, Venkatrama, B.E. Madras, A.M.I.C.E., of Madras, India.

OBITUARY

The Council regret to announce the deaths of Charles Simpson MCNEILL (Member); George Arthur MITCHELL (Retired Member); Christopher Frederick Edward BLACKMAN, Harry POVER (Associates); Norman Talbot FAGG, William SMITH, William WILD (Associate-Members).

EXAMINATIONS—JULY, 1952

OVERSEAS CENTRES

The Institution's Examinations were held in July, 1952, in Auckland, Baghdad, Beirut, Bombay, Bulawayo, Cairo, Calcutta, Cape Town, Christchurch, Colombo, Durban, Dunedin, East London (South Africa), Georgetown, Hong Kong, Jerusalem, Johannesburg, Kaduna (Nigeria), Karachi, Kirkuk, Kisumu (Kenya), Kuala Lumpur, Lagos, Lucknow, Melbourne, Mombasa, Nairobi Penang, Port Elizabeth (South Africa), Salisbury (Southern Rhodesia), San Francisco, Singapore, Sydney, Tel-Aviv, Wellington (N.Z.).

Forty-three candidates entered for the Graduateship Examination and 71 for the Associate-Membership Examination, making a total of 114. Of these, 30 passed the Graduateship Examination and 34 passed the Associate-Membership Examination.

The names of the successful candidates are :—

GRADUATESHIP EXAMINATION

ATKINSON, Frank Brian
BANERJEE, Subodh Chandra
BOSMAN, Edward Albert
BOTHMA, Kenneth Harold
CAREY, Dudley Layton
DAVID, Solomon Arulanandam
DHARWARKAR, Prabhakar Dhondopant
DU TOIT, George Alfred
DU TOIT, Pieter Schalk
EFFIONG, Maurice Okon
FOTHERINGHAM, John Henderson
FRANCIS, Herbert Bramwell
GARG, Banarsi Das
GOH JOON YAP
LONGRIGG, Thomas Derek
MEINEKE, Cyril Christopher
MORCOM, Peter John
NAMAN, Ihsan
READMAN, Ernest Victor
SINGH, Jattan
SINGH, Sarwan
STARR, Kenneth
SULLIVAN, Eric Harold
SUTHERLAND, Alexander John
TALEB, Jameel Nazih
TIEW YEW SENG
TOWSON, Valentine Geoffrey
TUCKER, Kenneth Charles
TURTON, Kenneth
TWADDLE, Ian Edward

ASSOCIATE-MEMBERSHIP EXAMINATION

ASHER, Peter John
AU SIK LING
BHANDARKAR, Prafullachandra Ganesh
BUCHAN, Alan James
BURTON, Robert Bentley
CHEN SHAN-CHUANG
CHRISTIE, Dennis
COHEN, Stanley Ernest
DAVIS, Elsbury John
DREW, Ryszard Bronislaw
EISINGER, Gerhard
GRAFF, Jacob Yehuda
GROYER, Edward
HALSALL, Robert
HANNA, Louis Demian
HILL, John Alexander
JOHANSEN, Cecil Clifton
LAING, Gordon Smith
LIAO CHUNG CHOU
LIGHTBAND, Michael Stuart
LOWE, Robert William
MIRVIS, Lionel
NAMAN, Ihsan
PARKYN, John Roy
PATWA, Sohrab Navalshaw
RUDD, Arthur Claude
SCHAFFNER, Robert Hastings
SHAH, Navinchandra Ambalal
SHIROMANI, Shriman Rai
SHVARTZ, David Itzhak
STANDER, Ruben
STOCH, Solly Gerald
WARNER, John Spearman
WHITFORD, Eric Thomas

PRIZE LIST

The Council have awarded the following prizes connection with the examinations held in July, 1952 :

ANDREWS PRIZE. (For the candidate who obtains t highest aggregate of marks in the Associate-Membersh Examination, passing in all subjects.)

Graham KINDER, of Stockport.

HUSBAND PRIZE. (For the candidate who takes t whole of the Associate-Membership Examination, pass in all subjects, and obtains the highest marks in the pap " Structural Engineering Design and Drawing.")

Michael Ernest BRYANT, of Beckenham, Kent.

WALLACE PREMIUM (SENIOR). (For the candida who takes the whole of the Associate-Membersh Examination, passes in all subjects, and obtains t highest marks in the paper " Theory of Structur (Advanced).")

Graham KINDER, of Stockport.

WALLACE PREMIUM (JUNIOR). (For the most successful candidate in the Graduateship Examination, passing in all subjects.)

Douglas James BATEMAN, of Chessington, Surrey.

EXAMINATIONS—JANUARY, 1953

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 6th and 7th, 1953 (Graduateship) and January 8th and 9th (Associate-Membership).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, January 8th, 1953

Joint Meeting with the Reinforced Concrete Association at 6 p.m., when Colonel A. R. Mais, O.B.E., T.D., and Mr. A. C. Little will give a paper on "The Construction of Eight Prestressed Concrete Tanks."

Thursday, January 22nd, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. B. A. E. Hiley, M.I.C.E. (Member of Council) will give a paper on "Electricity Generating Stations."

Thursday, February 12th, 1953

Ordinary Meeting at 6 p.m., when Dr. F. G. Thomas, M.I.C.E. (Member of Council) will give a paper on "The Strength of Brickwork."

Thursday, February 26th, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. P. L. Capper, T.D., M.Sc., A.M.I.C.E. (Member of Council) will give a paper on "Soil Mechanics in relation to Structural Engineering."

Thursday, March 12th, 1953

Joint Meeting with the Institute of Welding, at 6 p.m., when Mr. E. M. Lewis will give a paper on "The Construction of the new Testing Laboratory of the British Welding Research Association at Abington."

Thursday, March 26th, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Professor A. G. Pugsley, O.B.E., F.R.S., D.Sc., M.I.C.E., F.R.Ae.S. (Vice-President) will give a paper on "A Simple Theory of Suspension Bridges."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

JOURNAL CASES AND BINDING, 1952

A binding case can be supplied for the twelve issues of the Journal, January-December, 1952 (Vol. 30), price 1/6d. post free. The price for binding volumes is 2/6d. per volume, inclusive. This is for the half-leather binding which has been in use for some years.

It is requested that all parcels and Journals forwarded for binding should bear the name, address and rank of the member concerned. All volumes for binding must be despatched to the Institution by March 31st, 1953.

An Index will be included in all volumes bound. This Index will not be generally distributed, but members and others wishing to have a copy should apply to the Secretary.

DRURY MEDAL AWARD

The fourth competition for the above award will take place in 1953. The subject is the design of the structure of a new factory building. The material of construction is entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of structural arrangement. Economy in the use of steel is an important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary; envelopes to be marked in the top left-hand corner, "Drury Medal Award."

The closing date for the competition is October 1st, 1953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

MACLACHLAN LECTURE COMPETITION, 1953

The closing date for the receipt of entries for the next MacLachlan Lecture Competition will be Tuesday, March 31st, 1953. Particulars of the Competition are as follows :—

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering so long as in every second year the subject shall be confined to steel structures. (1953 is one of these years.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s. 0d.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer these sums to the Research Fund of the Institution.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1., on Tuesday, 27th January, 1953, at 6 p.m., when the President of the Institution, Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E., will give an address. It is hoped that as many members of the Section as possible will attend the meeting.

Hon. Secretary : C. Allen Brown, 43, Coolgardie Avenue, Highams Park, London, E.4.

INSTITUTION REPRESENTATIVES

AUSTRALIA

Victoria and New South Wales—A. J. Francis, M.Sc.(Eng.), B.Sc., A.M.I.C.E., A.M.I.Struct.E., Civil Engineering Dept., The University, Melbourne, N.3. Victoria, Australia.

Queensland—E. Snowden, B.Eng., A.M.I.C.E., A.M.I.Struct.E., c/o Department of Works, City Hall, Brisbane, Queensland, Australia.

BRITISH WEST INDIES

W. S. Sharp, A.M.I.Struct.E., Department of Works and Hydraulics, 1, Edward Street, Port of Spain, Trinidad.

CANADA

P. L. Pratley, D.Eng., M.I.C.E., M.I.Struct.E., 909, Drummond Building, St. Catherine Street West, Montreal, Quebec.

CEYLON

N. Wynne-Jones, C.B.E., F.R.I.B.A., M.I.Struct.E., P.W.D., Colombo, Ceylon.

EAST AFRICA

R. C. Marc, M.Sc., A.C.G.I., D.I.C., A.M.I.C.E., A.M.I.Struct.E., Public Works Department, Dar-es-Salaam.

HONG KONG

S. E. Faber, A.F.C., B.Sc., A.C.G.I., M.I.C.E., M.I.Struct.E., M.Am.Soc.C.E., 301, Princes Building, Chater Road, Hong Kong.

INDIA

Bombay—H. S. Batlivala, B.E., M.I.Struct.E., Concrete Association of India, 1, Queens Road, Bombay, 2, India.

H. Foster King, M.I.Struct.E., F.R.I.B.A., Chartered Bank Building, Fort, Bombay, 1.

Calcutta—J. Chambers, O.B.E., M.C., M.I.C.E., M.I.Struct.E., "Artistry House," 15, Park Street, Calcutta, 16.

Roorkee—Dr. C. A. Hart, T.D., D.Sc.(Eng.), M.I.C.E., F.R.I.C.S., A.M.I.Struct.E., Vice-Chancellor's Lodge, University of Roorkee, Roorkee, U.P., India.

ISRAEL

M. S. Gilutz, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., 24, Achad-Haam Street, Tel-Aviv.

NEW ZEALAND (NORTH ISLAND)

S. Irwin Crookes, Junr., B.A., M.I.Struct.E., Auckland University College, School of Architecture, Auckland, C.1.

NEW ZEALAND (SOUTH ISLAND)

C. W. Hamann, M.I.C.E., M.I.Struct.E., Messrs. Campbell and Hamann (Consulting Engineers), 136, Manchester Street, Christchurch, C.1.

PAKISTAN (WEST)

H. Holman, M.I.Struct.E., A.I.E.E., Messrs. Killick, Nixon & Co., Ltd., Bombay Company Building, Wallace Road, (P.O. Box 729), Karachi 2, Pakistan.

SINGAPORE

K. D. Mathewson, B.A.(Cantab.), M.I.C.E., M.I.Struct.E., c/o Messrs. Redpath, Brown & Co., Ltd., P.O. Box 648, Singapore.

SOUTHERN RHODESIA

K. G. Stevens, M.I.C.E., M.I.Struct.E., 721/2, Rhodelect House, P.O. Box 1669, Salisbury, Southern Rhodesia.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, January 13th, 1953

Joint Meeting with the Institute of Welding, Liverpool and District Branch, when the 1951 Larke Medal Paper on "Continuous Welded Structures, Abbey Works, Po Talbot" will be given by Mr. W. S. Atkins, B.S M.I.C.E., M.Inst.W., at the Liverpool College of Tec nology, at 7 p.m.

Wednesday, January 28th, 1952

Mr. Ronald Oates (Graduate) on "The Structur Design of the Medieval Cathedral."

Tuesday, February 24th, 1953

Mr. P. W. Rowe, B.Sc., Ph.D., A.M.I.C.E., "Developments in the Design of Sheet Pile Walls."

Thursday, March 26th, 1953

Joint Meeting with the Institution of Civil Enginee North Western Association. (Details to be announc later.)

All meetings, unless otherwise stated, will be held the Reynolds Hall, College of Technology, Mancheste at 6.30 p.m., preceded by tea at 5.45 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E 24, Kenwood Road, Stretford, Lancs.

MIDLAND COUNTIES' BRANCH

The following meetings have been arranged :—

Friday, January 23rd, 1953

Dr. K. Hajnal-Konyi, A.M.I.C.E., (Member) " Recent Applications of Shell Concrete Construction England and Wales."

Tuesday, February 17th, 1953

Mr. O. W. Jones, B.Sc., A.M.I.C.E. (Member), " Reinforced Concrete Foundations and Structures f Blast Furnaces and Materials and Handling Plant Shotton, Nr. Chester," at the King's Hall, Queen Stree Derby, at 7 p.m.

Friday, February 27th, 1953

Mr. N. T. Grant on "Experiences with Concrete."

Friday, March 27th, 1953

Details to be announced.

All meetings, unless otherwise stated, will be held the James Watt Memorial Institute, Birmingham, 6 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES' BRANCH
GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Friday, January 30th, 1953

"Further Recent Midland Structures."

Descriptions of rebuilding of Marshall & Snelgroves an C. & A. Modes (War Damage); Birmingham Technic College (Steel Frame Building); Grosvenor Hous Birmingham (Foundations).

Tuesday, March 31st, 1953

Annual General Meeting.

All meetings to be held in the James Watt Memori Institute, Birmingham, at 7 p.m.

Hon. Secretary : F. G. Fletcher, 60, Brean Avenu South Yardley, Birmingham, 26.

NORTHERN COUNTIES' BRANCH

The following meetings have been arranged :—

Tuesday, January 6th, 1953

Joint Meeting with the Institution of Civil Enginee at Middlesborough.

Wednesday, January 14th, 1953

Joint Meeting with the Northern Architectur Association at Newcastle.

Tuesday, February 3rd, 1953

Mr. B. A. E. Hiley, M.I.C.E., (Member of Council) " Electricity Generating Stations," at Middlesborough

Wednesday, February 4th, 1953

The above meeting will be repeated at Newcastle.

Tuesday, March 3rd, 1953
Ladies Guest Night at Middlesborough.

Wednesday, March 4th, 1953
Ladies Guest Night at Newcastle.

Tuesday, March 31st, 1953
Annual General Meeting.

All meetings commence at 6.30 p.m., the Middlesborough Meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and those at Newcastle in the Neville Hall, near the Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesborough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Monday, January 19th, 1953
Joint Meeting with the Institution of Civil Engineers, Northern Ireland Association. Mr. Harold E. Sidwell, M.Sc., A.M.I.C.E., on " Reinforced Concrete Building in Brazil," at Queen's University, Belfast, at 5.45 p.m.

Tuesday, February 10th, 1953
Annual Dinner and Social Function. Visit of the President and Secretary of the Institution. At Northern Counties Hotel, Belfast, at 6.30 p.m.

Tuesday, March 3rd, 1953
Mr. J. C. Malcolmson, B.Sc. (Member), on " Recent Developments in Prefabricated Concrete Structures."

Tuesday, March 31st, 1953
Annual General Meeting.

All meetings, unless otherwise stated, will be held in the College of Technology, Belfast, at 6.45 p.m., preceded by tea at the Overseas League premises, Wellington Place, Belfast at 6 p.m.

Hon. Secretary : S. Duckworth, M.I.Struct.E., " Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meeting has been arranged :—

Wednesday, March 18th, 1953
Mr. H. Nelson, B.Sc., A.R.T.C., on " Tension Members in Engineering Structures " at the Royal Technical College, George Street, Glasgow, at 6 p.m.

Hon. Secretary : D. G. Drummond, B.Sc., M.I.Struct.E. A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH WESTERN COUNTIES' BRANCH

The following meetings have been arranged :—

Friday, January 23rd, 1953
Mr. Leslie Richardson, A.M.I.C.E. (Associate Member), on "Construction of Two Power Stations in the South-West."

Friday, February 6th, 1953
Mr. F. R. Bullen, B.Sc., M.I.C.E. (Member of Council), on " Unusual Design for a Large Constructional Shop."

All meetings will be held at the Duke of Cornwall Hotel, Millbay, Plymouth, at 7 p.m.

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10-12 Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, January 21st, 1953
Junior Members Evening at Swansea.

Wednesday, February 11th, 1953
Mr. D. Manolopoulos (Member), on " Report on the 4th Congress of the International Association for Bridge and Structural Engineering," at Swansea.

Tuesday, February 17th, 1953
The above meeting will be repeated at Cardiff.

Wednesday, March 11th, 1953
Mr. J. R. M. Ewen, B.Sc., on " Tubular Structures," at Swansea.

Tuesday, March 17th, 1953
The above meeting will be repeated at Cardiff.

Friday, March 27th, 1953
Branch Annual Dinner at Swansea.

Meetings in Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel, at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES' BRANCH

At the Opening Meeting of the Session held in the University of Bristol on Friday, October 10th, members had the pleasure of meeting the President and Secretary of the Institution. The Chairman, Mr. E. N. Underwood B.Sc., M.I.C.E. (Member), presented his address " Problems in Practice," and a vote of thanks was given by Mr. J. W. Lorraine (Member).

After the meeting, the Chairman and members of the Branch entertained the President and Secretary at an informal dinner in the Royal Hotel, Bristol.

The following meetings have been arranged :—

Friday, January 2nd, 1953
Mr. F. R. Bullen, B.Sc., M.I.C.E. (Member of Council), on " Unusual Design for a Large Constructional Shop."

Friday, February 6th, 1953
Mr. F. G. Clarke (Associate Member), on " Some Local Contracts and Welded Steelwork for a Bus Garage."

Wednesday, February 18th, 1953
Annual Dinner.

Thursday, March 5th, 1953
Combined Meeting with the Institution of Civil Engineers. Mr. L. Richardson, A.M.I.C.E. (Associate Member), on " Plymouth ' B ' Power Station," at the Reception Room, University of Bristol, at 5.30 p.m. Tea at 5 p.m.

All meetings, unless otherwise stated, will be held in the University of Bristol, Geology Lecture Theatre (entrance University Road), at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 39, Effingham Road, St. Andrew's Park, Bristol, 6.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, January 21st, 1953
Mr. Donovan H. Lee, B.Sc., M.I.C.E., M.I.Mech.E., M.Am.Soc.C.E., (Member of Council), on " Design of Prestressed Concrete."

Wednesday, February 18th, 1953
Dr. S. Mackey, M.E., B.Sc., A.M.I.C.E.I., (Associate Member), on " Secondary Stresses in Steel Bridge Girders."

Wednesday, March 18th, 1953
Mr. H. E. Manning, B.Sc., M.I.C.E. (Member of Council), on " Report on the Papers given on Concrete Shell Roofs in London, July, 1952."

All meetings will be held at the University, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted at the City Engineer's Department, City Hall, Johannesburg. 'Phone : 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed

Continuous Rigid Portal Frames

To the Editor of THE STRUCTURAL ENGINEER

Sir,—Mr. Bolton has recently pointed out in your columns that in an article dealing with the solution of continuous ridged portal frames, no example with a north light type of roof or with unsymmetrical load on the rafters was given, and goes on to say that the calculations of fixed-end constraints and of the effect of unit operations for such conditions makes a solution much more difficult.

Lest this comment be misunderstood, I would draw attention to the fact that lack of symmetry does not affect the essential attack in any way. It is only necessary to do some preliminary arithmetic in order to start the solution. On the attached figure the appropriate calculations for an unsymmetrical frame carrying a side load on the roof are set out (by the method of slope-deflection, though alternative methods may be used at the choice of the designer) and it is hoped that these will clarify the point to which Mr. Bolton draws attention.

Beeston, Notts. Yours, etc.,
September 15th, 1952. E. MARKLAND

To the Editor of THE STRUCTURAL ENGINEER

Sir,—We were interested to read Mr. Springfield's letter in the October number of THE STRUCTURAL ENGINEER.

The errors reported in the model analysis solution seem far greater than we have been accustomed to obtain on models tested in the Department of Building and Structural Engineering here, and likely to bring the method into disrepute.

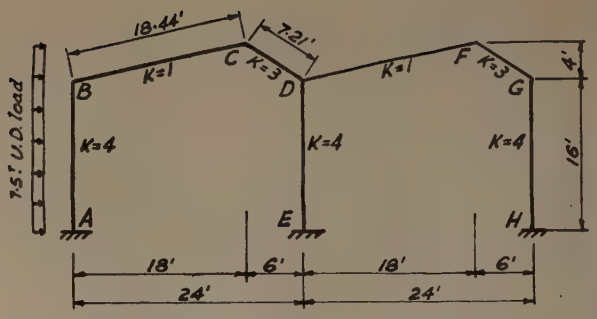
The apparatus used here is the large deformation apparatus as developed by Messrs. W. S. Atkins and Partners. Using this apparatus determinations can be made directly for moments at the valley joints by applying internal rotations at these points. This avoids the difference of large numbers effect when the moments are calculated from the redundants at the stanchion feet.

An investigation into Mr. Springfield's method of using models of distorted scales leads to the conclusion that it is wrong in two important respects.

1. Mr. Markland showed (THE STRUCTURAL ENGINEER, May, 1952) that the horizontal scale of a ridge may be distorted so far as the horizontal and rotational displacement of one end is concerned. Unfortunately, this is not true for vertical displacements as it would have to be for the method to be used to determine vertical reactions as done by Mr. Springfield.

2. The distorted ridge offers the same restraints for rotation and horizontal displacement as the true scale ridge does to the stanchions. Therefore the deflections of the stanchions for horizontal and rotational displacements will be the same in both cases, and true influence lines can be obtained for forces on the stanchions. The deflections on the rafter in the distorted ridge are not however the same as those of a true scale model, and the influence lines cannot be obtained directly for this part of the structure as done by Mr. Springfield. They could, however, be deduced from the rotations and displacements at the valley joints.

Yours, etc.,
Manchester. W. MERCHANT
October 9th, 1952. A. BOLTON



For roof member BCD: Consider unit rotation at B, $\Delta\theta_B = 1$.

Slope-deflection equations:

$$M_{BC} = 2(2+\theta_C)$$
$$M_{CB} = 2(1+2\theta_C)$$
$$M_{CD} = 6(2\theta_C)$$
$$M_{DC} = 6(\theta_C)$$

$\theta_C = -0.125$

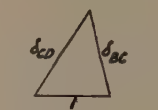
$M_{BC} = +3.75$, $M_{CB} = +1.50$, $M_{CD} = -1.50$, $M_{DC} = -0.75$

Similarly for unit rotation at D.

Consider unit sway without rotation at B, Δu

From Williot deflection diagram

$\delta_{BC} = -1.152$ $\delta_{CD} = +1.352$



Deflection diagram

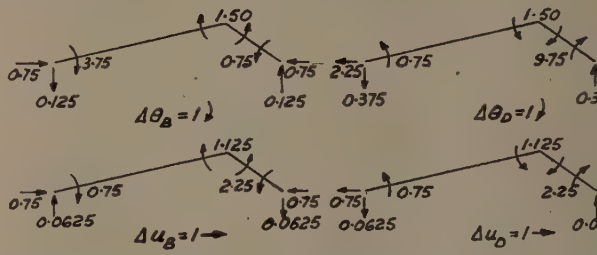
Deflected shape

$$M_{BC} = 2\left(\theta_C + \frac{3\times1.152}{18.44}\right)$$
$$M_{CB} = 2\left(2\theta_C + \frac{3\times1.152}{18.44}\right)$$
$$M_{CD} = 6\left(2\theta_C - \frac{3\times1.352}{7.21}\right)$$
$$M_{DC} = 6\left(\theta_C - \frac{3\times1.352}{7.21}\right)$$

$\theta_C = 0.1875$

$M_{BC} = +0.750$, $M_{CB} = +1.125$, $M_{CD} = -1.125$, $M_{DC} = -2.250$

Summarizing effects of unit operations (the reactions being obtain from known moments by statics



Fixed-end moments at B & D due to applied load may be obtaine treating BC & CD as spans of a continuous beam since BC cannot sway without either B or D.

F.E.M.s. on BC are $\frac{WL}{12} = \frac{1.5\times4}{12} = 0.500$

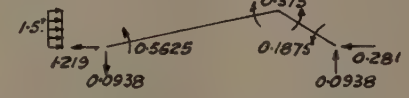
Slope-deflection equations:

$$M_{BC} = 2(\theta_C) - 0.500$$
$$M_{CB} = 2(2\theta_C) + 0.500$$
$$M_{CD} = 6(2\theta_C)$$
$$M_{DC} = 6(\theta_C)$$

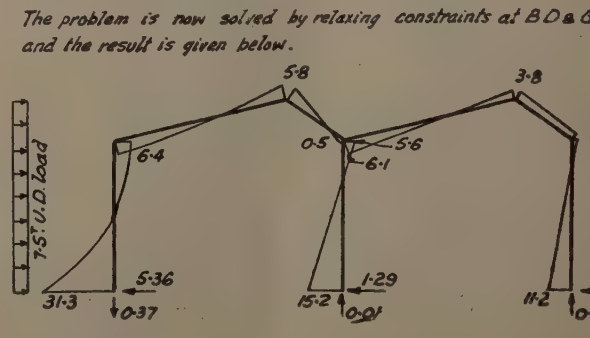
$\theta_C = -0.03125$

$M_{BC} = -0.5625$, $M_{CB} = +0.375$, $M_{CD} = -0.375$, $M_{DC} = -0.1875$

Hence the following fixed-end conditions for BCD subject to applied lo



The problem is now solved by relaxing constraints at B D & G and the result is given below.



The Strength of Brickwork*

By F. G. Thomas, Ph.D., B.Sc., M.I.C.E., M.I.Struct.E. (Member of Council)

Summary

In the last twenty-five years large numbers of brick alls and piers have been tested at the Building Research ation. The results of these tests are examined in lation to the design recommendations given in Code of ractice CP111 (1948) "Structural Recommendations r Load-bearing Walls." As a result, modifications are ggested which lead to a more reasonable distribution load factor than that obtained by adopting the present ode recommendations.

Experimental data are given also on the behaviour panel walls subjected to lateral loading, and on the iffening and strengthening effect of such panels on the sistance of structural frameworks against racking ads.

Introduction

Brickwork has been used for thousands of years but ne scientific investigation of its strength is comparavely recent. In 1887-8 a committee of the American ociety of Civil Engineers conducted what appears to e the first series of tests, made on thirty-three piers to etermine the strengths of brickwork built in three types f brick with both cement mortars and lime mortar, and ntroducing the effect of varying slenderness.

A few years later, a committee of the Royal Institute f British Architects conducted a similar series of pier ests using four types of brick with either cement mortar r lime mortar, but adopting a constant slenderness ratio f four throughout. A comprehensive report on this nvestigation was published by the R.I.B.A. in 1905.

During the last 25 years a considerable amount of ata has been steadily obtained on the strength of piers nd walls, particularly in America, at the Universities f Ohio and Illinois and at the Bureau of Standards, and n this country at the Building Research Station. During his period, tests have been made on over 500 piers and valls at the Building Research Station, and although nany problems have yet to be solved, enough has been earnt already to give a sound basis for "designed" rickwork. It is interesting to note that one of the arliest publications of the Building Research Board was report on the stability of thin walls issued in 1921. This report dealt with experiments carried out by Dr. Oscar Faber for the D.S.I.R. Building Materials Research Committee, which preceded the Building Research Board. The main conclusion from those xperiments was that with truly axial loading, the reduction in strength with increasing slenderness was remarkbly small, except when lime mortar was used.

The results of some of the early B.R.S. investigations n the strength of brickwork were published in Building Research Special Report No. 22, "Mechanical Properties f Bricks and Brickwork Masonry," by Glanville and Barnett, issued in 1934. A fairly comprehensive survey f the work was made in a paper by Davey and Thomas, resented to the Institution of Civil Engineers in 1950. Other papers have appeared from time to time in echnical journals, dealing with particular aspects of the vork, e.g., the paper by Thomas and Simms on the trength of reinforced brick masonry beams in THE TRUCTURAL ENGINEER for July, 1939.

In considering the form of the present paper, the writer had particularly in mind that the British Standard Code of Practice CP111 (1948), "Structural Recommendations for Loadbearing Walls," which was prepared by a committee convened by the Institution, is shortly due for review and, if necessary, revision. Some of the recommendations contained in this Code have, therefore, been considered in the light of the experimental data available and suggestions are made as to improvements which could be made if the Code is revised.

One difficulty in preparing a design specification for load-bearing brickwork is the great variability that can occur both in the strength of the materials and the quality of the workmanship. When there are many factors influencing the strength of a structure and the effect of each factor is known only within wide limits, there is sometimes a tendency for engineers to adopt limiting "safe" assumptions for each factor ; the result may be that the overall load factor is unnecessarily high for most practical examples. This tendency, in the writer's opinion, must be guarded against if brick structures are to be designed economically, but the best method of dealing with variability has yet to be determined.

It is important also for the research worker to realise that some of the variations that occur in practice are absent from tests in the laboratory and are, in fact, almost impossible to reproduce there. The effects of differences in workmanship, for example, are not known with any certainty. The writer made one series of experiments in which a group of piers was built by each of six bricklayers, with a view to finding the human factor in the strengths subsequently attained by the brickwork. The results did not give the data required ; each of the craftsmen, with an eye on the testing machine in the background, produced the best brickwork he could, and there were no significant differences in strength that could be attributed to variation in workmanship. At the U.S. Bureau of Standards some years ago the effect of workmanship was studied more successfully by employing two independent bricklayers to construct similar walls. One man worked by piecework and was not supervised ; the other was employed at an hourly rate and was carefully supervised. The walls built by the latter man were from 30 to 60 per cent. stronger than those built by the unsupervised bricklayer.

Not only will poor workmanship often lead to lower basic strengths of brickwork but it may also affect the form of the relationships between strength and such factors as slenderness or eccentricity of loading.

Another difficulty in the design of brick structures is the uncertainty as to what allowance to make for deterioration of the brickwork, particularly of the mortar joints. At present, the effect is rather arbitrarily included in the consideration of the load factor to be adopted, but it may be possible in the future to differentiate between various types of brickwork with a view to eliminating partially the allowance for deterioration in circumstances where such deterioration is relatively unimportant.

Basic Stresses for the Design of Brickwork

In the design of concrete structures, the permissible stresses in the concrete are dependent on the cube strength of the concrete ; similarly, it would be reason-

*Paper to be read before the Institution of Structural Engineers t 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 12th, 1952, at 6 p.m.

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able to base the design of brickwork on the strength of a cube of brickwork, using the appropriate brick and mortar to give the required cube strength, which would be checked during the course of the work. The making and testing of brickwork cubes would be much more inconvenient, however, than for concrete and in view of the likely variability of the results of such tests it would be necessary to make a large number of test samples. In the Code of Practice, therefore, brickwork design is based on permissible stresses tabulated for

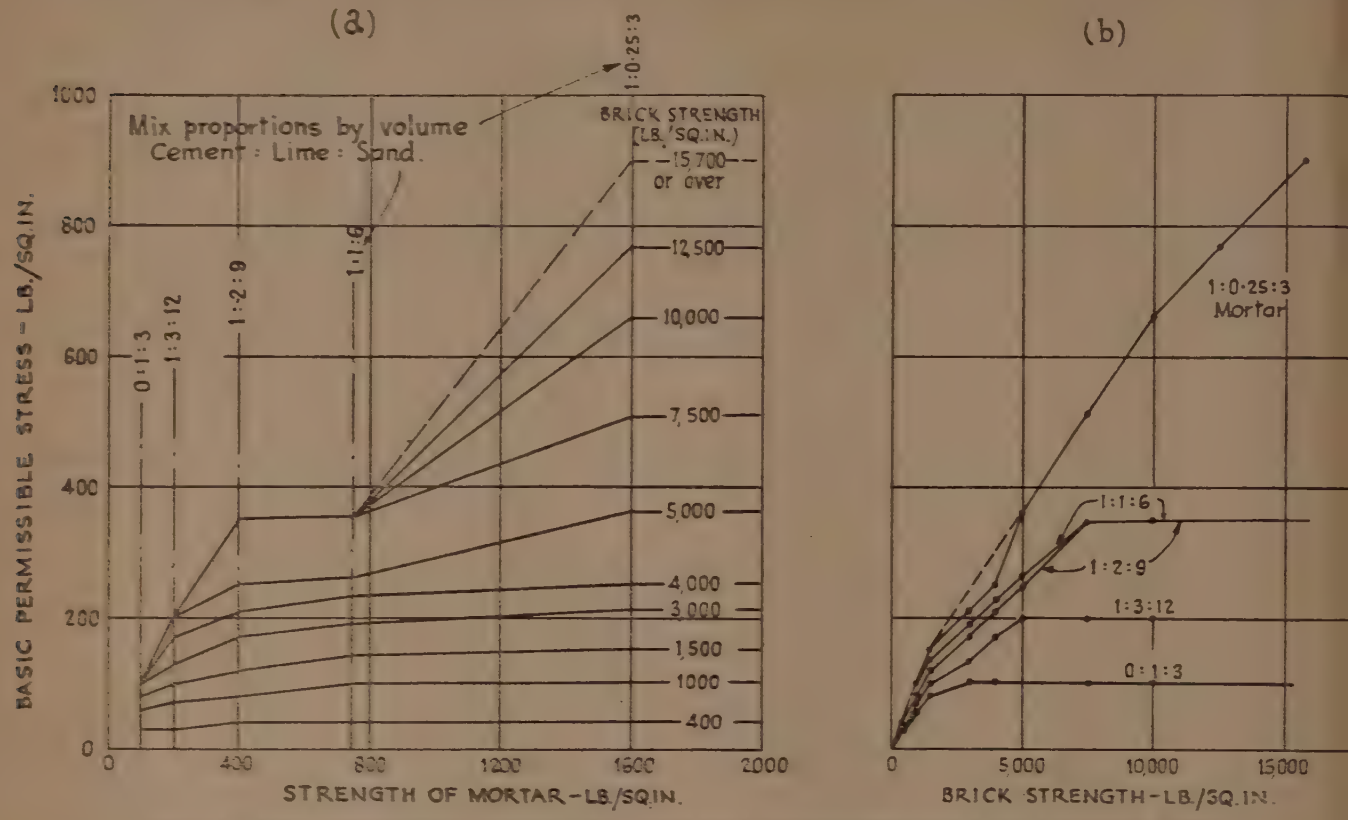


Fig. 1.—Basic permissible stresses for brickwork recommended by Code CP111.101 (1948)

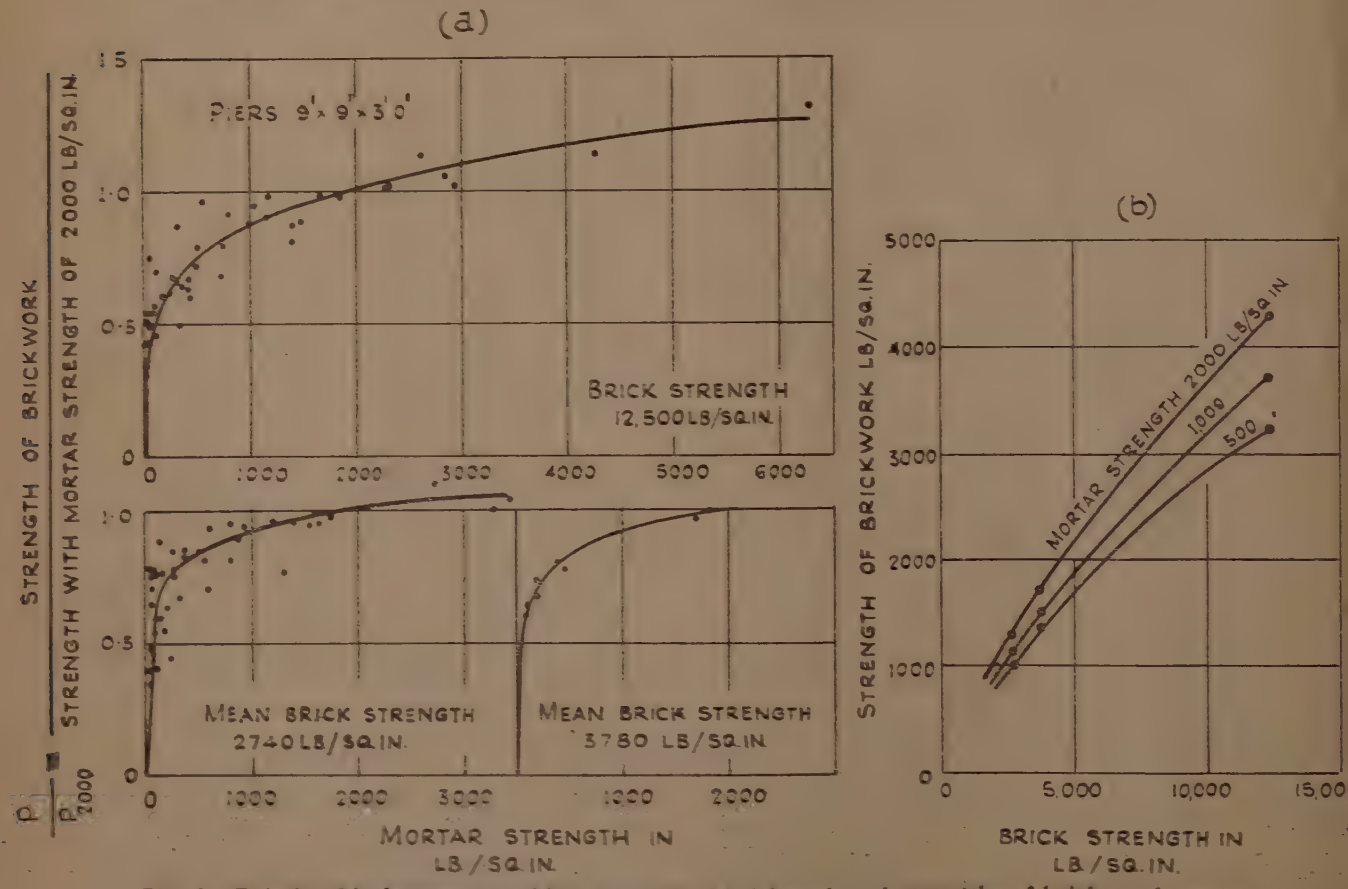


Fig. 2.—Relationship between crushing strength of brickwork and strengths of bricks and mortar

brick pier or wall of slenderness ratio equal to unity, these stresses being related to the strength of brick and type of mortar used.

The basic stresses are shown diagrammatically in Fig. 1. In Fig. 1 (a) the lines for each brick strength show the permissible stresses only where they cross the vertical lines representing the various mortar mixes, since the Code does not permit interpolation for mix proportions intermediate between those for which stresses are tabulated. (Interpolation for intermediate brick strengths in Fig. 1 (b) is, however, allowed.) is imposed by the Code on the brickwork stress, and, rather illogically perhaps, the same limit of 350 lb. per sq. in. has been specified for both a 1 : 1 : 6 and a 1 : 2 : 9 mortar.

In order to obtain data on the effect of varying the strengths of brick and mortar on the strength of brickwork, a very large number of tests have been made at the Building Research Station on piers 9 in. square in section and 3 ft. high. Some of the results obtained are shown in Fig. 2. In Fig. 2 (a), the strengths of the piers have been plotted as proportions of the strength

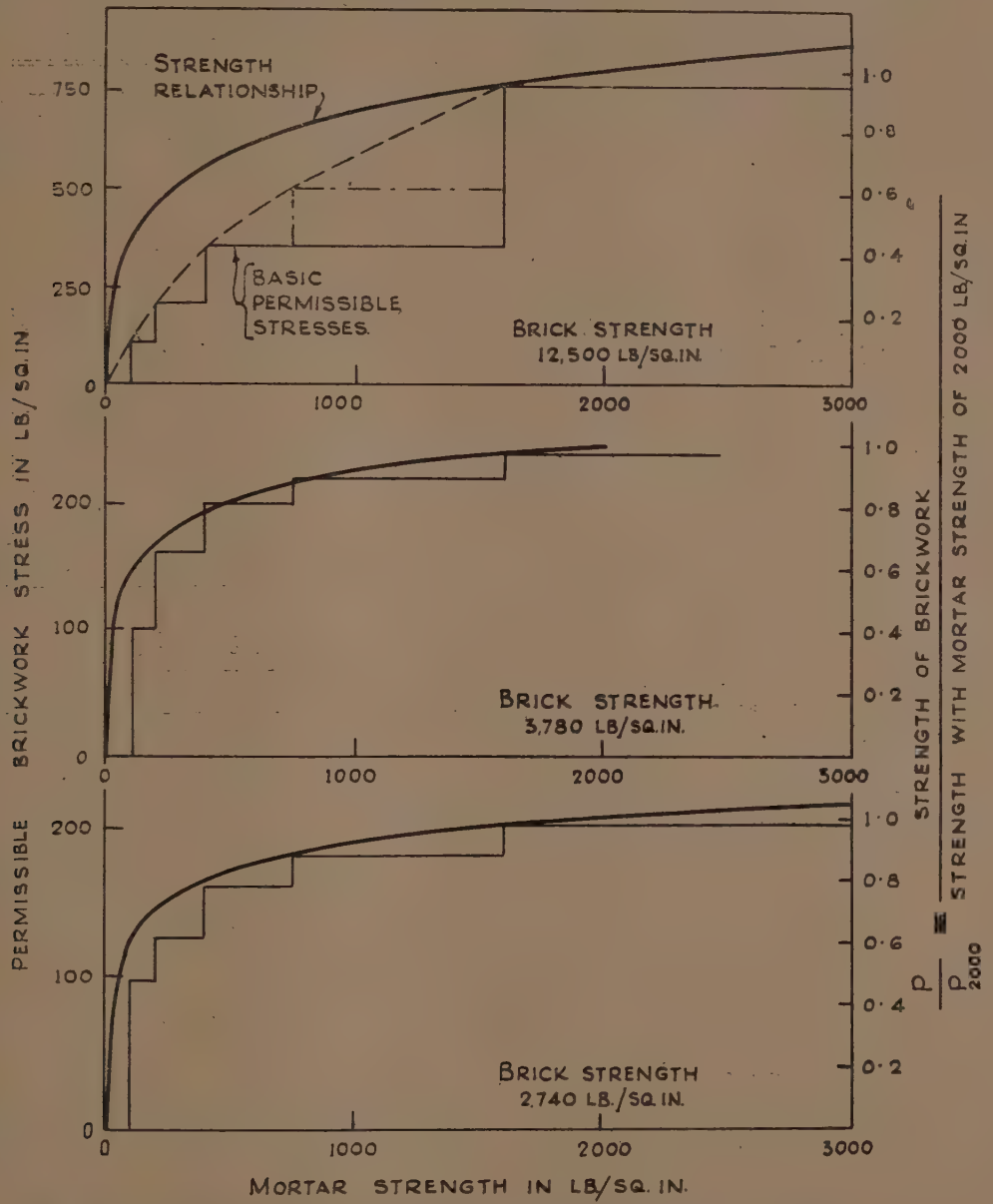

STRENGTH RELATIONSHIP
BASIC PERMISSIBLE STRESSES
BRICK STRENGTH 12,500 LB/SQ.IN.
BRICK STRENGTH 3,780 LB/SQ.IN.
BRICK STRENGTH 2,740 LB./SQ.IN.
PERMISSIBLE BRICKWORK STRESS IN LB./SQ.IN.
STRENGTH OF BRICKWORK / STRENGTH WITH MORTAR STRENGTH OF 2000 LB/SQ.IN
p / p2000
MORTAR STRENGTH IN LB/SQ.IN.

Fig. 3.—Comparison of form of relationship between brickwork strength and mortar strength, with the permissible stresses in Code CP111.101

Certain features of Fig. 1 are of particular interest. For brick strengths above 1,000 lb. per sq. in. the adoption of a stronger mortar is usually accompanied by an increase in the stress permitted in the brickwork. For bricks of 7,500 lb. per sq. in. or stronger, however, this rule does not hold for a change from a 1 : 2 : 9* mortar to a 1 : 1 : 6 mortar, the permissible brickwork stresses being the same for the two mixes. This curious feature is due to the fact that, for each type of mortar, a limit corresponding to a mortar strength of 2,000 lb. per sq. in. ; in this way, the results of several series of tests, with somewhat varying mortars and bricks, have been combined to indicate the general form of the relationship between the strength of the brickwork and that of the mortar. The relationships shown for the three strengths of brick are of interest in indicating that the value of increasing the mortar strength, in terms of the strength of brickwork resulting, becomes steadily less ; and also, that for any chosen proportion of the optimum brickwork strength the required mortar strength increases with the strength of brick used.

*Throughout this paper, mortar proportions are given as cement : lime : sand, by volume.

Fig. 2 (b) shows how the brickwork strength varies with the strength of brick used. The exact form of the curves cannot be established with the data available, but it is at any rate obvious that, for mortar strengths of 500 lb. per sq. in. or more, the strength of the brickwork is appreciably increased by using stronger bricks up to at least 12,500 lb. per sq. in. It seems, therefore, that there need be no limiting stress for any particular mortar, as has been adopted in the Code of Practice, see Fig. 1 (b).

The empirical relationships between the brickwork strength and the strength of the mortar, given in Fig. 2 (a), are compared in Fig. 3 with the Code recommendations for the basic permissible stresses. In this figure, the scales are chosen so that the two curves, being compared, touch at the point corresponding to a mortar strength of 1,600 lb. per sq. in. (the average strength of a 1 : 0.25 : 3 mortar in the experiments being considered) ; and the increases in permissible stress occur at the average mortar strengths corresponding to the mortar proportions referred to in the Code.

It will be seen that, for the two lower brick strengths, the Code "stepped" relationship is of a reasonable form compared with the empirical relationship. There is a tendency for the permissible stress to fall more rapidly with decreasing mortar strength than the actual reduction in strength of the brickwork but this might well be justified on the grounds of greater variability in strength when a weak mortar is used. For the brickwork using 12,500 lb. per sq. in. bricks, the Code relationship is very different from the actual strength curve and it is evident that an increase in stress when using a 1 : 1 : 6 mortar, as shown by the chain-dotted lines in Fig. 3, is the least modification that should be made to the Code. It also seems desirable to adjust the relative values of permissible stress for the various mortars, but the figure is not in a form which indicates whether the stresses should be increased for brickwork with the weaker mortar or if the stresses should be reduced for brickwork with the stronger mortar. Guidance on this point can be obtained by reference to the load factors, i.e. the ratios of the failing loads of the piers to the permissible stress corresponding to the brickwork used and the slenderness of the piers tested. The load factors of the piers to which Fig. 3 relates are given in Table 1, and it will be observed that the load factor for the brickwork with high-strength bricks and a strong mortar is similar to that when the lower-strength bricks are used, but the load factor increases very considerably when weak mortars are associated with the high-strength bricks. The conclusion appears to be that with high-strength bricks (10,000 lb. per sq. in.) the permissible stresses might well be increased by 50 per cent. with a 1 : 3 : 12 mortar and 75 per cent. with a 0 : 1 : 3 mortar. The table also indicates the desirability of increasing the stresses when a 1 : 1 : 6 mortar is used with a high-strength brick, and an increase of 30 per cent. appears reasonable.

A curious feature of the Code recommendations for brickwork with a 1 : 0.25 : 3 mortar is the divergence of the permissible stresses for brick strengths of 3,000 and 4,000 lb. per sq. in. from the linear relationship between allowable stress and brick strength for the range of 1,500 to 10,000 lb. per sq. in. bricks, as shown in Fig. 1 (b). It seems probable that this divergence was not intended by the Code drafting committee and it is evident that in any revision of the Code the inconsistency should be removed. The values should clearly be :—

Brick strength	: 3,000, 4,000 lb. per sq. in.
Permissible stress in brickwork	: 240, 300 lb. per sq. in.

The recommendations of Code CP111.101 (1948) for the basic stresses are given in Table 2 together with the modifications which seem reasonable in the light of the experimental data available.

The Strength of Slender Walls and Piers

(a) *Axially loaded piers.* The strength of any compression member is to some extent dependent on its slenderness. The ultimate load may be controlled either by the onset of elastic instability, or by over-

TABLE 1.—Load Factors for Piers with Various Brick and Mortar Strengths

Average Brick Strength —lb. per sq. in.	Load Factor for Brickwork using a mortar mix* of :				
	1 : 0.25 : 3	1 : 1 : 6	1 : 2 : 9	1 : 3 : 12	0 : 1 : 3
2,740	5.2	5.6	5.2	6.6	5.0
	5.4	5.9	6.2	7.2	5.2
	5.7	6.4	6.2	8.1	6.6
	5.9	6.7	9.7	8.2	7.1
	6.3	7.7		8.3	7.2
	6.7	9.5		8.6	7.3
	7.2			8.8	7.5
	7.7			10.1	7.9
	8.5			10.2	8.9
	9.1			10.4	9.5
				11.8	10.1
				11.9	11.4
					12.8
					12.8
Averages :	6.8	7.0	6.8	9.2	8.5
3,780	7.3	6.4	6.4	7.3	
	8.0	7.0	6.8	7.4	
Averages :	7.7	6.7	6.6	7.4	
12,500	5.0	9.8	7.4	11.8	11.9
	5.4	10.7	8.2	14.0	13.2
	5.5	10.8	8.7	15.5	14.5
	5.7	11.8	9.6	18.9	17.0
	6.0	12.0	9.8	19.2	20.2
	6.4	13.5	11.1	21.0	20.5
	6.6		12.5	29.2	24.5
	6.7				26.4
	6.8				27.0
	7.1				31.7
	7.8				34.0
	8.5				35.6
	9.3				
Averages :	6.7	11.4	9.6	18.5	23.0

*The actual mix proportions were not always those stated, but were classified according to the Code requirements as a basis for permissible stress determination. The proportions are for cement : lime : sand, by volume.

stressing of the material of which the member is constructed. With brickwork, elastic instability is unlikely to be of importance except possibly with very slender walls built with lime mortar, or when the wall is "dry-built" using fibrous board or other comparatively compressible jointing material. The reduction in strength of slender piers usually arises from the effects of variability in the properties of the brickwork, or accidental eccentricities of loading and lack of straightness of the axis of the pier, and other defects arising from imperfect workmanship ; all these effects tend to become of greater significance as the slenderness of the pier is increased.

The results of tests on axially loaded piers of either 9 in. or 13½ in. square cross-section and for slenderness

tios (height divided by thickness) up to 18, are shown Fig. 4. Four types of brickwork were used, and it ill be observed from the figure that the greatest reduc- on in strength with increasing slenderness was obtained hen a weak brick was used with a weak mortar. The sults are shown as " strength reduction coefficients," iat is, as proportions of the strength of a similar pier with a slenderness ratio of unity. Slight extrapolation of the results was necessary for this purpose, as the shortest piers had a slenderness ratio of 2.4. Each of the solid circles in the figure is a common point for half the results, one for the 9 in. piers and the other for the 13½ in. piers.

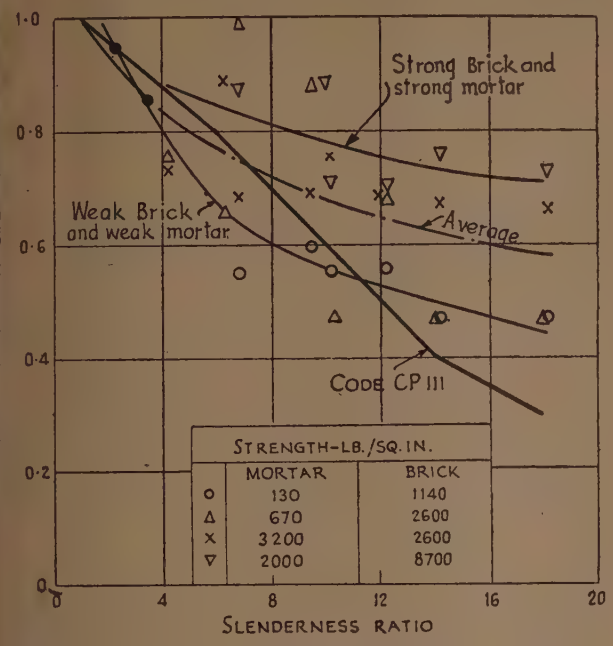


Fig. 4.—Strengths of axially-loaded piers of various slenderness ratios, as a proportion of the strength of a pier of slenderness ratio of unity

With this method of presentation, direct comparison is possible with the " reduction factors for slenderness ratio " given in Table 2 of Code CP111.101 (1948), and the Code recommendations are indicated on Fig. 4. As the slenderness ratio increases, the Code leads to a greater margin of safety. If it is agreed that variability of the results about the mean is reasonably left to be allowed for in a general load factor, then the average curve shown in the figure could be adopted in place of the Code relationship, when considering axially-loaded piers. For design purposes it would probably be impracticable to introduce modifications to the general relationship in terms of the use of weak or strong mortars.

It should be noted that in the tests, the piers were tested between knife edges, so that no directional restraint was afforded to their ends. In practice, some directional restraint is almost always present but the Code of Practice, in the opinion of the writer quite rightly, makes no special allowance for such restraint ; nevertheless, the strength is thereby somewhat increased.

(b) *Axially-loaded walls.* The Code assumes that the effects of slenderness are not so serious for walls as for piers, since, although the reduction factors for slenderness apply to both, the slenderness ratio of a wall is taken to be only three-quarters of that of a pier of the same height and thickness. There appears to be some justification for this on statistical grounds, since the

TABLE 2.—Maximum permissible uniformly distributed compressive stress (at and after the stated times) on masonry members with a slenderness ratio of unity. Table 1 of Code CP111.101 (1948) with suggested modifications in brackets)

Description of Mortar	Mix (Parts by volume) not weaker than			Hardening Time after completion of work (Days)†	Maximum uniformly distributed stress in lb./sq. in. Corresponding to units whose crushing strength (in lb./sq. in.) is :‡								
	Cement	Lime	Sand		400	1000	1500	3000	4000	5000	7500	10,000	10,000 + x
(i) Cement	1	0-¼*	3	7	40	100	150	210 (240)	250 (300)	360	510	660	660+0.042 x but not more than 900
(ii) Cement-Lime	1	1	6	14	40	100	140	190	230	260 (270)	350 (360)	350 (450)	350 (450+0.025 x but not more than 600)
(iii) Cement-Lime	1	2	9	14	40	80	120	170	210	250 (240)	350 (300)	350 (360)	350 (360+0.017 x but not more than 450)
(iv) Cement-Lime	1	3	12	14	30	70	100	130 (140)	170	200	200 (250)	200 (300)	200 (300+0.014 x but not more than 350)
(v) Hydraulic Lime	—	1	2	14	30	70	100	130 (140)	170	200	200 (250)	200 (300)	200 (300+0.014 x but not more than 350)
(vi) Non-Hydraulic	—	1	3	28**	30	60	80	100	100 (110)	100 (120)	100 (145)	100 (170)	100 (170+0.007 x but not more than 200)

*The inclusion of lime in cement mortars is optional, see Clause 202 (a) of Code CP111.101.
†These periods should be increased by the full amount of any time during which the air temperature remains below 40°F., plus half the amount of any time during which the temperature is between 40°F. and 50°F.
‡Linear interpolation is permissible for units whose crushing strengths are intermediate between those given in the Table.
**A longer period should ensue where hardening conditions are not very favourable.

various imperfections, which cause reduction of strength in slender members, will in a stretch of walling be less likely to produce a general weakening of the brickwork than might occur in a pier, particularly a small one.

Unfortunately, no series of tests has been made on the strength of walls loaded axially through knife-edges, for direct comparison with the results for piers. The following results for walls 4½ in. thick and 4 ft. 6 in. long bedded at their ends on the plates of the testing machine are however of interest. Medium-strength bricks were used with a 1 : 1 : 6 mortar.

Slenderness ratio (As defined by Code)	7.5	10.5	13.5	18	19.5	21	22.5
Failing load of pier (tons)	128 136	107	158 134	151 148	155	137	132

From these figures it will be seen that there is no significant reduction in strength with increased slenderness over two-thirds of the range covered by the Code.

(c) *Eccentrically-loaded piers.* When a pier is eccentrically loaded, the pier deflects laterally and the eccentricity of loading is thus increased. This effect may be very pronounced for a tall pier, and particularly so as failure is approached and the lateral stiffness of the pier is reduced as a result of cracking. It follows therefore that the strength reductions for slender piers with eccentric loading are greater than for axially loaded piers ; and that the use in design of a single set of "reduction factors for slenderness ratio" for all brickwork must lead to a lower load factor when eccentricity of load is present on tall piers.

Tests have been made at the Building Research Station on over a hundred piers, either 9 in. or 13½ in. square in section, to determine the relationship between strength, slenderness, and eccentricity of loading. Some results of the tests on 9-inch piers are shown in Fig. 5, in which are plotted the load factors obtained by dividing the failing loads by the permissible working load as calculated in accordance with Code CP111.101.

Reference to Fig. 5 shows that the load factor for axially loaded piers increases with slenderness and that there is a similar increase for piers with an eccentricity of loading equal to one-twelfth of the pier thickness. The calculation of load factors for the eccentrically-loaded piers includes the allowance of 25 per cent. increase in stress given in Clause 304 of Code CP111.101, and the results shown in Fig. 5 for small eccentricities of loading afford justification for this allowance. As the eccentricity of loading increases, the rise in load factor with the more slender piers becomes less ; and for an eccentricity equal to one-third of the pier thickness, the load factor is approximately independent of the slenderness ratio, with a tendency for this factor to be somewhat lower at a slenderness ratio of about 10 than for smaller or larger ratios.

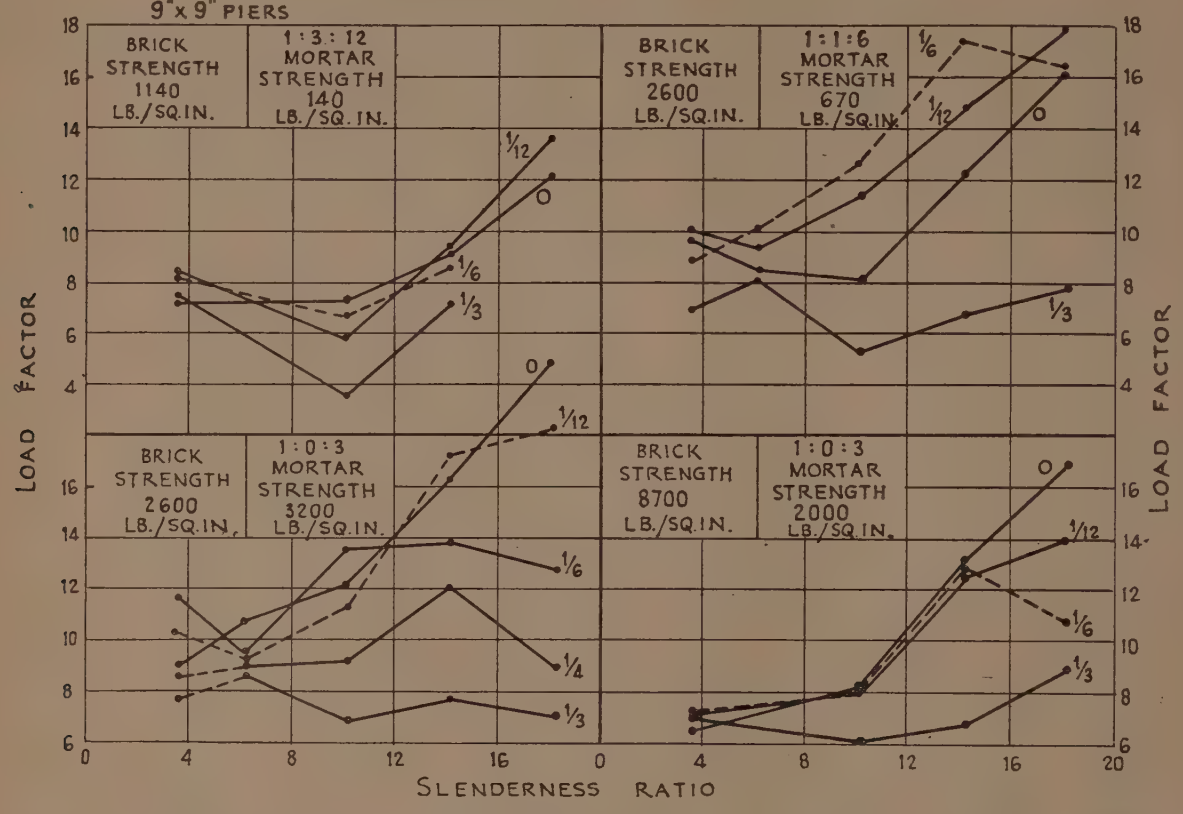


Fig. 5.—Variation of load factor of brick piers with slenderness ratio and eccentricity of loading. (The figures against the curves give the eccentricity as a fraction of the pier thickness)

Comparison of the four diagrams in Fig. 5 for the different types of brickwork does not indicate any major difference of behaviour, and it appears reasonable to use the same reduction factors in design for all brickwork, introducing the effect of eccentricity of loading on these factors, as shown tentatively in Fig. 6. No experimental data are available for eccentricities ε of loading greater than one-third of the pier thickness t_p so that the limiting curve for $\varepsilon = t_p/2$ is uncertain. However, in practice it will usually be wise to avoid eccentricities greater than $t_p/3$ in view of the considerable increase in compressive stress that accompanies loading approaching the edge of the brickwork.

(d) *Eccentrically loaded walls.* Tests have been made on a series of 4½ in. brick walls 4 ft. 6 in. long with slenderness ratios ranging from 7.5 to 22.5, with two eccentricities of loading equal to one-ninth and two-

nths of the wall thickness. The load factors for ese walls are given in Table 3 for two assumptions as the stress reduction factors for slenderness :

(i) using Table 2 of Code CP111.101 ;

(ii) using the values given in Fig. 6.

will be observed from Table 3 that although the use of g. 6 leads to less variation in the load factor for the

TABLE 3.—Load Factors for Eccentrically Loaded Walls

Slenderness Ratio as defined by Code	Load Factors			
	$\epsilon/t = 1/9$*		$\epsilon/t = 2/9$	
	Based on Code reductions for slenderness	Based on Figure 6	Based on Code reductions for slenderness	Based on Figure 6
7·5	9.0	9.2	8.4	8.8
10.5	11.3	10.3	9.8	9.6
13·5	12.7	9.6	12.2	10.4
16.5	16.8	10.9	10.0	7.7
18.0	13.5	8.4	9.1	6.8
19.5	13.3	7·9	9.8	7·3
21.0	15·4	8.9	9.1	6.9
22.5	14.1	7·9	7·9	5·9

* ϵ = eccentricity of loading.
t = thickness of wall.

maller eccentricity, the Code recommendations are ightly to be preferred, from this point of view, for the alls with the greater eccentricity and slenderness atios of 18 or more. It may be possible to modify ig. 6 to avoid its apparent deficiencies in terms of very ender walls with eccentric loading but insufficient data

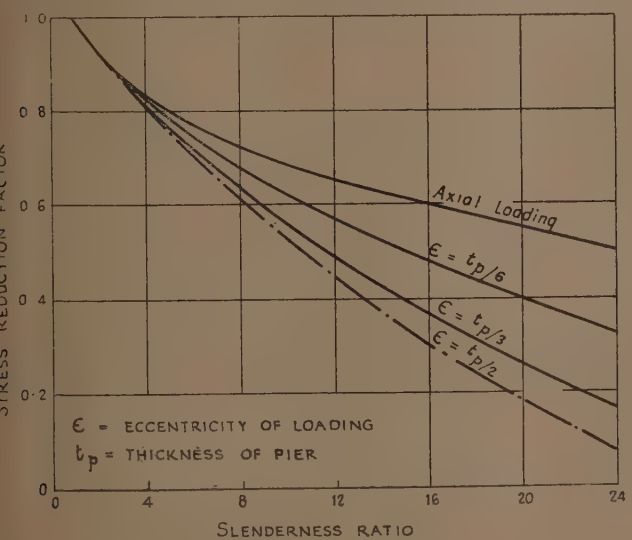


Fig. 6.—Effect of eccentricity of loading on the reduction coefficients to be assumed for slender piers. (c.f. Table 2 of Code CP111.101)

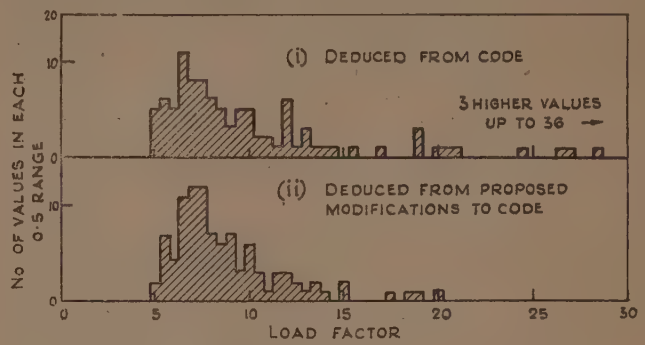
re available to do this with much accuracy. However, his is not a matter of great practical importance at resent since the Code limits the slenderness ratio of alls to 18, except for domestic dwellings ; and for ousing a greater slenderness for brick walls will be very are (a 4½ in. wall with a slenderness ratio of 18 would e 9 ft. high from floor to floor).

ffect of the Proposed Modifications to Design Stresses on the Variation in Load Factor

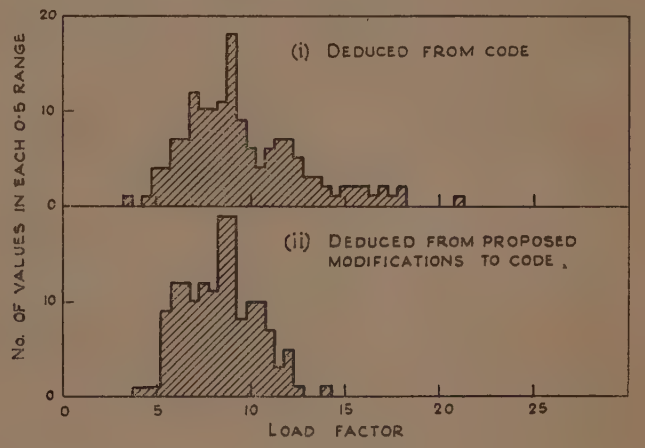
Frequency diagrams are given in Figs. 7 and 8 to dicate the variability of the load factor for brickwork (i) esigned in accordance with Code CP111.101 ; and (ii), introducing the modifications to the basic stresses, as indicated in Table 2 and to the stress reduction factors for slenderness, as shown in Fig. 6.

In Fig. 7 (a) the results apply to the series of tests covering a wide range of brick and mortar strengths, and the revised diagram shows essentially the effect of the modifications in basic stress given in Table 2 ; it also includes a general increase in load factor of 5 per cent., since the stress reduction factor for an axially-loaded 3 ft. × 9 ft. × 9 in. pier, is only 0.84 in Fig. 6 as compared with 0.88 in Table 2 of Code CP111.101.

In Fig. 7 (b) the results apply to the series of tests covering a wide range of slenderness and eccentricity of loading and the revised diagram shows essentially the



(a) RESULTS FOR SERIES WITH VARYING BRICK AND MORTAR STRENGTHS



(b) RESULTS FOR SERIES WITH VARYING SLENDERNESS AND ECCENTRICITY OF LOADING

Fig. 7.—Load factors for brick piers, showing effect of proposed modifications to Code CP111.101

effect of the proposed revision of the stress reductions for slender members ; it also includes the small effect of the suggested increase in basic stress for brickwork using a 1 : 0.25 : 3 mortar with a brick strength of 3,000 lb. per sq. in.

The results given in Fig. 8 give a general picture of the effect of the proposed modifications, and include not only the two series to which Fig. 7 refers, but also other tests on walls and piers that have been made at the Building Research Station from time to time.

These diagrams indicate that the effect of the proposed modifications is not to produce a *general* reduction in load factor but to eliminate some of the very high load factors for which there is no justification. On the available evidence, the writer feels that there should not be an overall reduction in load factor and that the form of the revised diagram in Fig. 8 is satisfactory. It seems also from this diagram that the load factor of six recommended in Clause 803 of CP113, when test panels are built to determine the strength of wall construction, is reasonable. Several of the variables for which allowance

is normally included in the load factor are eliminated when a wall is tested and it is therefore appropriate that the load factor to be associated with such a test on *brickwork* should approach the lower limit of the factors associated with design based on strengths of *bricks* and on proportions (not strengths) of mortars.

Cavity Walls

Very little information is available on the structural behaviour of cavity walls and consequently the Code

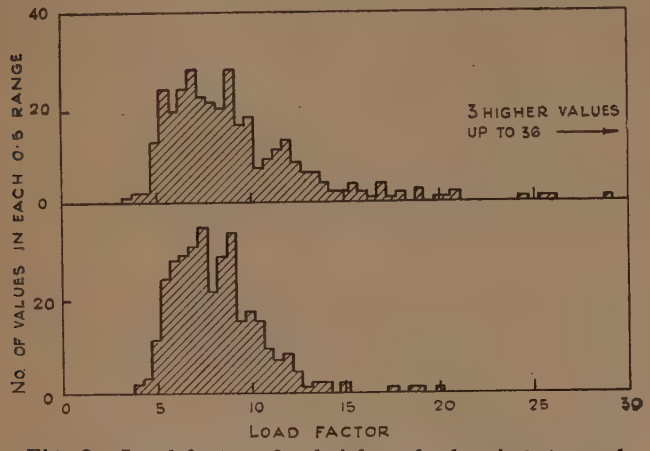


Fig. 8.—Load factors for brickwork showing general effect of proposed modifications to Code CP111.101

gives little guidance on their strength and stability. There is often confusion between the terms " strength " and " stability " and in fact many people treat them as being synonymous. In the writer's view they should properly be taken to mean two distinct properties which are, so to speak, " internal " and " external " properties of the structure (or structural element). Strength is a physical property by virtue of the resistance of the material, of which the structure is made, to forces which tend to over-*stress* it ; stability implies a resistance to large *movement* of the structure as a whole.

The differentiation can be seen for a cavity wall by reference to the sketches in Fig. 9. In the left-hand diagram, case (a), the loads from the first floor and the roof are carried by the inner leaf only and the design is such that these loads are axially applied to this leaf. The inner leaf is well supported at ground, first floor and roof levels ; and no question of instability arises. A that is necessary is to make certain that it is stron enough to carry the loads. The outer leaf is, howeve in a different category. It supports no vertical loa other than its own weight and its strength is relative unimportant ; it is merely necessary to see that it not tipped over by wind forces. It is clear that th problem is essentially one of strength for the inner le and stability for the outer leaf. From practical exper ence the rules for spacing of wall ties given in the Co are known to be satisfactory for stabilising the out leaf, at any rate for two or three-storey dwellings.

In sketch (b) in Fig. 9, it is assumed that deflectio of the first floor and an eccentric load from the ro cause bending of the inner leaf as shown. This arrang ment is worse for this leaf than that in sketch (a) an the wall would need to be stronger ; there is still n question of instability, however. As regards the out leaf, it is pulled or pushed by the wall ties, that are pu in to stabilise it, so that its shape is exactly the sam as that of the inner leaf. For this example, therefor the outer leaf must have strength as well as stability in fact the need for strength arises out of the provisio for stability.

The graphs in Fig. 9 show how well light wire ties ca ensure that the bending of the two leaves is the sam The results shown apply to tests on 11 inch cavity wal with only one leaf loaded at an eccentricity of 1 inc from the axis of this leaf. The eccentricity was som times arranged so that the ties tended to pull the u loaded leaf, whilst in other tests it was arranged so th the ties were put into compression. The ties were of th " butterfly " type of 9 S.W.G. steel wire or of 12 S.W.G copper wire, the latter complying with the minimu requirements of B.S. 1243 " Metal Wall Ties." It clear from these and other similar results that for cavi walls with two similar leaves, any bending momen induced by eccentric loading on either or both leav will be shared equally by them ; for dissimilar leav the moments will be shared in proportion to the sti nesses (EI) of the leaves.

This effect provides some justification for the ru for the effective thickness for a cavity wall given Clause 306 of Code CP111 :—

$$t = 2/3\ (t_o - w_c)$$

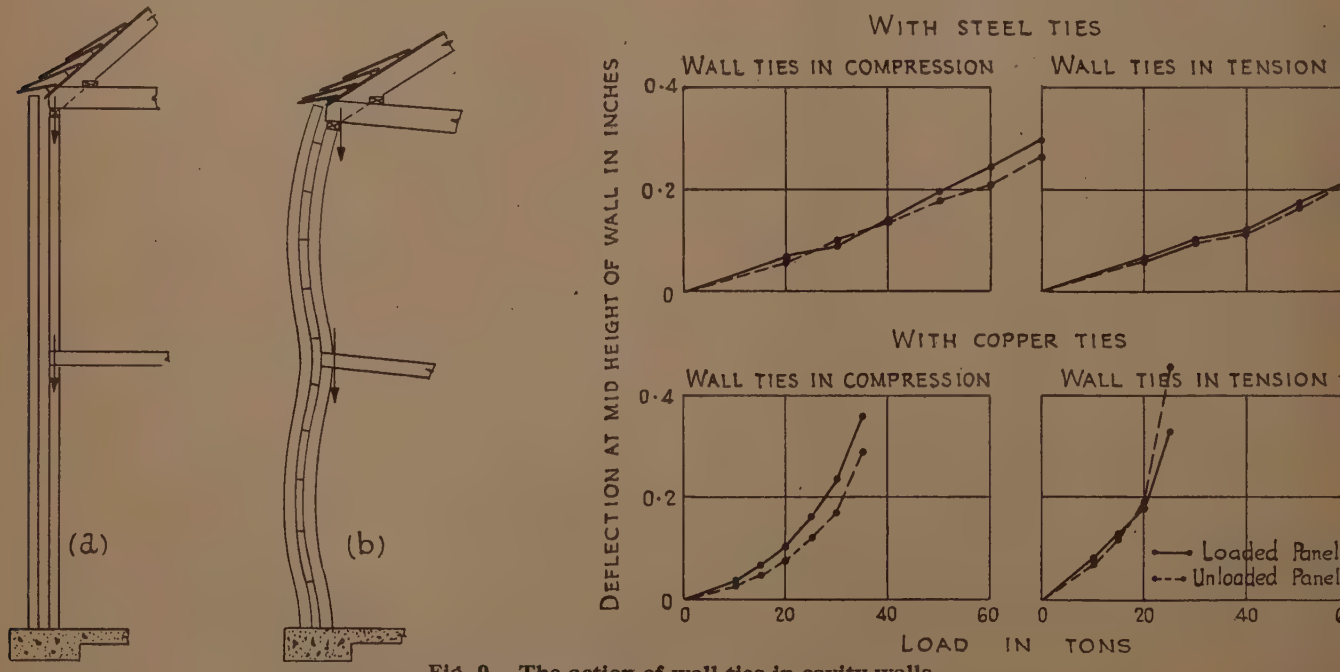


Fig. 9.—The action of wall ties in cavity walls

where t_o is the overall thickness of wall, including width of cavity, and w_c is the width of the cavity.

It has already been suggested that the reduction in strength of slender piers and walls arises from the amplification of the effects of accidental defects and eccentricities in the brickwork. When two walls are joined together by wall ties to form cavity construction, the consequent equalising of lateral deflections means that the equivalent accidental eccentricity for the combined system is the average of that for the two separate leaves. A simple statistical exercise shows that this average will more frequently tend to zero than the values for individual walls, so that for assumedly axial external loading on each leaf of a cavity wall, although the strength will vary between twice the lower and higher limits of load for each leaf tested separately, the average strength of the cavity construction will be appreciably higher than twice the average load for single walls.

The Code formula is a simple device to lead to increased stresses for cavity construction as compared with those for single walls of the same thickness as one leaf of the cavity wall ; and this formula appears to be entirely satisfactory when both leaves are equally loaded. If, however, one leaf is loaded less than the other, the value of their inter-connection by wall ties becomes less in terms of the *ultimate strength* of the system, although still effective in redistributing bending stresses at working loads. In the limit, if one leaf only is loaded, and eccentrically so, the bending induced in the unloaded leaf may lead to cracking of this leaf before the loaded leaf has attained the load it could have carried alone. In this example, theoretically, the cracked leaf would be stabilised by the loaded leaf, which would then bear all the bending moment imposed by the eccentric load and continue to take further load as though the other leaf were not present. In fact, the stress increase in the loaded leaf on cracking of the unloaded leaf may possibly lead to premature collapse of the whole system. For the tests to which Fig. 9 refers, there was apparently a slight disadvantage, so far as ultimate strength was concerned, in tying an unloaded leaf to the loaded leaf when the steel wire ties were used ; with the copper ties, there was apparently an advantage with the ties in compression and a disadvantage with the ties in tension. However, the differences were probably not significant.

It is evident from the foregoing that it is desirable in practice to load both leaves of a cavity wall if possible. In house construction, for example, although the inner leaf should wholly carry the first floor load, the roof load should be shared by both leaves.

The Code formula for the effective thickness of cavity walls is not limited to systems in which the same structural units are used for the two leaves. In house construction it is becoming common practice to use lightweight blocks for the inner leaf of external walls, with a 4½ inch brick outer leaf. In this system the inner leaf is the more heavily loaded but it receives considerable strengthening by virtue of its connection to the stiffer outer leaf ; probably for this example the Code formula errs somewhat on the safe side. In one series of tests, for example, it was found that a 3-inch leaf of lightweight clinker blocks, when loaded with an eccentricity of ½ inch, was 20 per cent. stronger when tied to a 4½-in. brick leaf than when tested alone with no eccentricity of load.

If, however, the inner leaf was constructed of much stronger material than that used for the outer leaf, and the inner leaf carried all the imposed load, adoption of the Code formula might well lead to a reduced load factor, and it would appear desirable to introduce a requirement in the Code that, when the load is carried by one leaf only, the formula should not be applied if that leaf is constructed of stronger structural units than the unloaded leaf.

Spacing of wall ties. Experience and tests have indicated that, with the usual spacing of ties as recommended by the Code, there is a reasonable margin of safety against structural failure of the ties, when used in the cavity walls of houses, even for the lightest wire ties permitted by B.S. 1243. For multi-storey buildings, however, strip metal ties are preferable and their efficiency can be deduced from the tests on cavity walls under lateral loading, described later in this paper. It will be observed from Fig. 10 (b) that pressures up to about 4 lb. per sq. in. were borne by 11 in. cavity walls ; and assuming that half of this pressure was transmitted to the rear leaf by the standard strip ties (three per square yard), each tie was carrying over 800 lb. in compression.

Where the only lateral pressure to be considered is that due to wind, it is evident that the standard strip metal ties at the usual spacing of 3 ft. horizontally and 1 ft. 6 in. vertically will be adequate for the external walls of all buildings and there may be some justification for reducing the number of ties somewhat. However, it should be remembered that the ties tend to deteriorate with time and also that some weakening of the bond between the tie and the mortar in which it is embedded

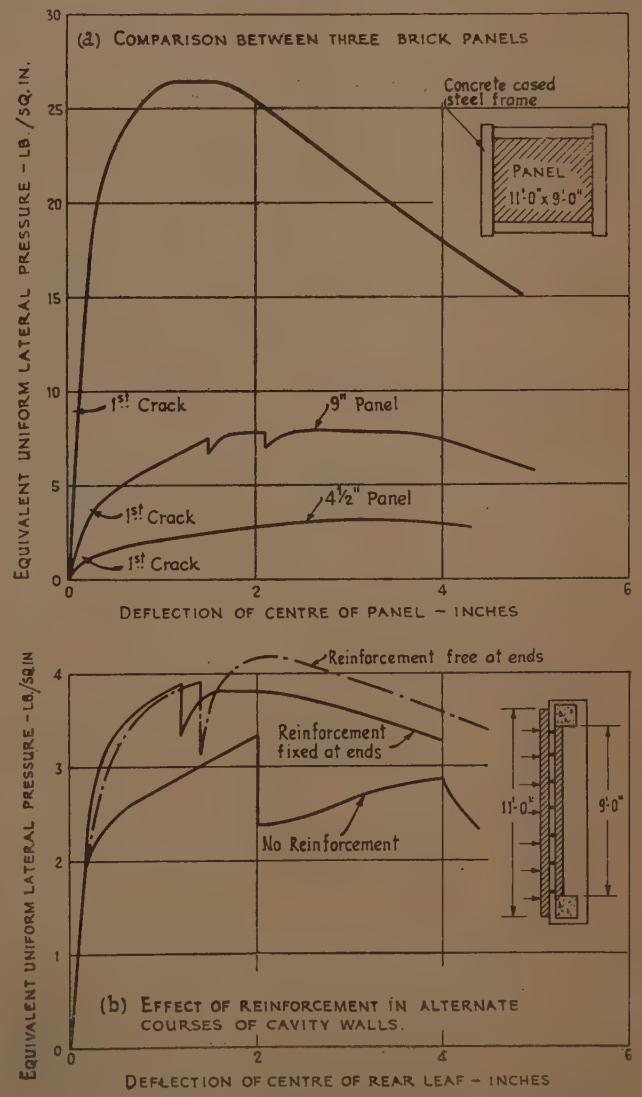

(a) COMPARISON BETWEEN THREE BRICK PANELS
EQUIVALENT UNIFORM LATERAL PRESSURE - LB./SQ. IN.
DEFLECTION OF CENTRE OF PANEL - INCHES
Concrete cased steel frame
PANEL 11'-0" x 9'-0"
1st Crack
9" Panel
4½" Panel
1st Crack
1st Crack
Reinforcement free at ends
Reinforcement fixed at ends
No Reinforcement
11'-0"
9'-0"
(b) EFFECT OF REINFORCEMENT IN ALTERNATE COURSES OF CAVITY WALLS.
EQUIVALENT UNIFORM LATERAL PRESSURE - LB./SQ.IN.
DEFLECTION OF CENTRE OF REAR LEAF - INCHES

Fig. 10.—Load-deflection relationships for walls under uniform lateral pressure

may result from continuous differential thermal and shrinkage movements. These movements may have serious effects in large stretches of cavity walling, particularly where the inner leaf is built in small sections into the framework of a building, whilst the outer leaf is continuous for the full height and length of the building. It seems preferable to divide the outer leaf also into sections, so that any section does not extend beyond, say, three storeys in height and 100 ft. in length.

Special difficulties arise with party walls of cavity construction, in that the requirements of fire resistance and sound insulation can lead to designs which are not structurally desirable. In order to avoid spread of fire, metal "hangers" are sometimes used to carry the ends of floor joists instead of building the joists into the party wall; this arrangement leads to considerable eccentricity of load on the wall. Further, the designer, wishing to minimise the transmission of sound across the party wall, reduces the number of ties to a minimum and adopts the lightest wire tie permitted by B.S. 1243. For cavity party walls in houses, hangers should be avoided unless the leaves of the cavity are at least of 4-inch thickness; the ends of joists can instead be supported in suitable sockets of incombustible material built into the wall. Where floor loads are not carried on the party wall, the number of ties can be reduced to one per square yard.

Lateral Strength of Walls

All walls, whether designed to support vertical load or not, are liable to be subjected to lateral loading, as for example from wind or from materials stacked against them. As the tensile strength of brickwork is low and variable, cracking may occur for even small lateral loads if the wall is thin or if insufficient support is given to the perimeter of the wall.

Cracking in itself may not lead to collapse, however, for the cracked wall may be stabilised by the action of any vertical loading it supports. Alternatively, if restraint to edge movement is afforded by the rest of the structure the wall may resist further lateral load by acting as a flat dome. This latter action can occur when a brick panel is built into or against a steel framework and tests have been made at the Building Research Station, at the request of the Ministry of Works, to examine the lateral strength of such panels. Some of the results are shown in Fig. 10.

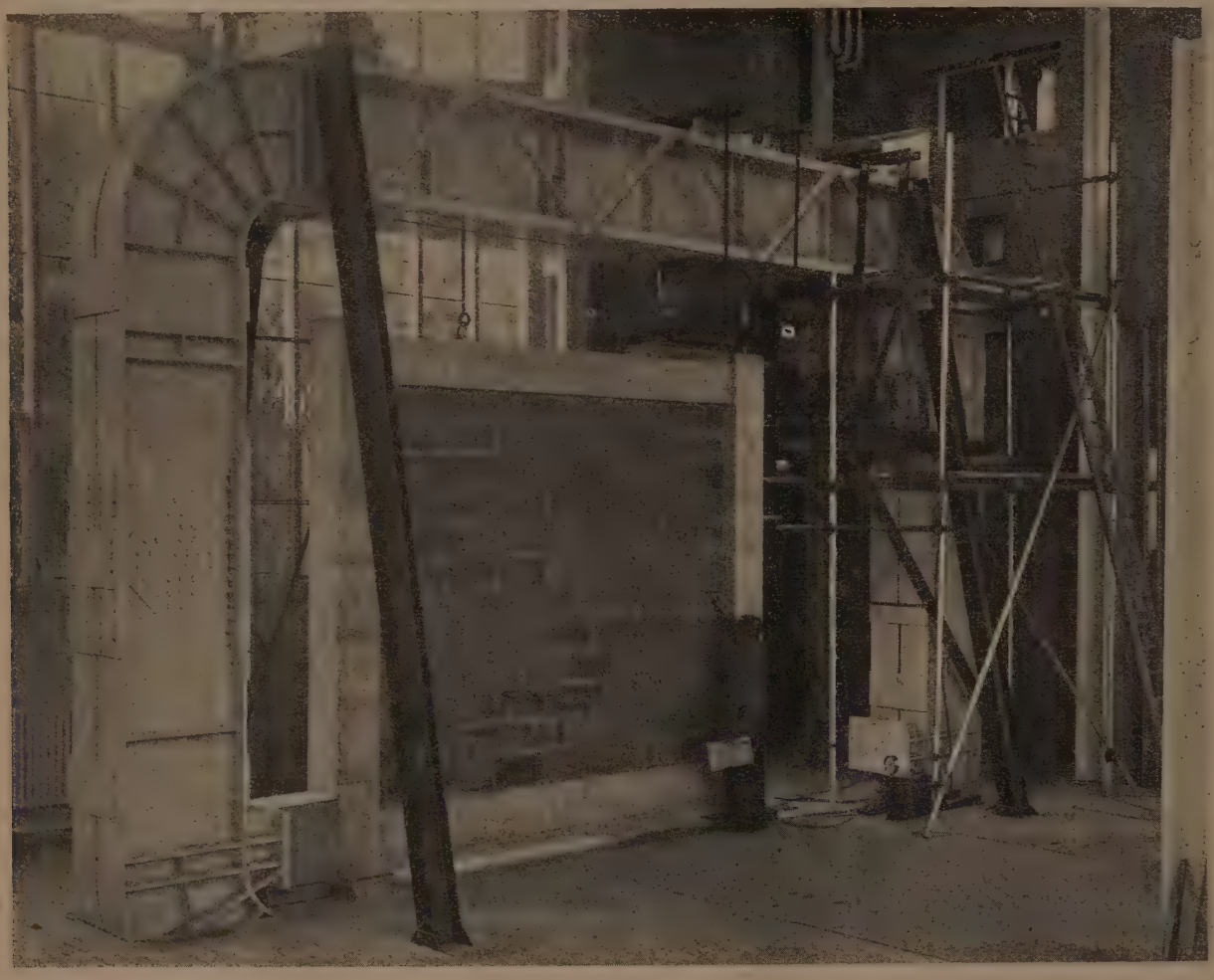


Fig. 11.—Test on panel in racking test-frame

The framed structure was represented by a steel frame consisting of two 10 in. × 8 in. stanchions and two 10 in. × 4½ in. beams. The members were encased in concrete and the distances, centre to centre, of stanchions and beams were 12 ft. and 10 ft. respectively. The brickwork of the test walls was built in Fletton bricks with a 1 : 0.25 : 3 mortar, as recommended in the Code for reinforced brickwork. The mortar had a compressive strength of about 1,600 lb. per sq. in. and a tensile strength of 190 lb. per sq. in. at the time of testing the walls, three weeks after construction.

Some single walls were built into the frame as a panel filling, all four edges being "buttered" with mortar to the adjacent surfaces of the concrete encasement of the frame members. The inner leaves of some cavity walls were similarly built in the frame whilst the sides of the outer leaves were bedded against the encased stanchions; the top edges were partially fixed by struts

d clamps to simulate the continuity that would be esent in practice.

In some of the walls, reinforcement was placed in ternate courses of each leaf of the brickwork. This inforcement consisted of the usual two-wire type often nployed in 4½-inch brick walls with a wire diameter 0.145 inch and an ultimate strength of 39 tons per . in. Sometimes the wires were not held at their ds ; in other walls, the wires left projecting from the ds of the wall were fixed to the stanchions.

The walls were subjected to lateral loading at 16 points aced so that the conditions were not very different om those that would be obtained with uniform lateral essure. The load was applied by means of four ydraulic jacks through a distributing system of steel ists ; in order to avoid local failure at the load points, mber pads 7 inches square were interposed between ie distribution joists and the brickwork.

The results for the single walls in Fig. 10 (a) show that nsiderable increase of load occurred after the initial acking of the walls. The 4½-inch wall resisted a maximum pressure of 3 lb. per sq. in. before signs of ushing and splitting appeared along all four edges. he initial failure of the 9-inch wall was due to a break-own of bond between the mortar at the bottom of the all and the encased beam. This led to a sudden drop load of 10 per cent., which, however, was re-imposed ; nd although a similar partial bond failure occurred at ie top of the wall, it was not until the deflection at the ntre of the wall had increased to nearly 4 inches that ie damage resulted in a reduction in the lateral load iat could be withstood. The 13½-in. wall cracked at a essure of about 8 lb. per sq. in., but continued to carry creasing load until at a pressure of over 26 lb. per . in. crushing began at the top edge. The maximum ad was maintained until the central deflection of the all was 1¾ inches, when bond and shear failure at and ear the bottom of the wall was accompanied by the ening of a central vertical crack from the bottom to ie middle of the wall.

The results shown in Fig. 10 (b) indicate the value of using light reinforcement in the horizontal courses of the leaves of a cavity wall. Both the load at first observed cracks and the maximum load were increased by the reinforcement ; there was however no apparent advantage in anchoring the wires to the steel frame. In both the reinforced cavity walls some of the wires fractured as the wall was deflected further after the maximum load had been passed. In the unreinforced cavity wall a sudden drop in load occurred at a pressure of about 3¼ lb. per sq. in. when a portion of the front leaf was forced into the cavity ; a partial recovery followed until spalling of the bricks at the edges of the rear leaf caused a steady decrease in the load-bearing capacity of the wall.

It was realised that in practice the efficiency of the process of "buttering" the sides and top of a panel wall to obtain a good mortar bond between the brickwork and the encased frame might vary considerably. In order to find out what might be the effect of *very* bad workmanship in this respect one reinforced 4½-in. wall was tested with a gap at the sides and top of the wall, but the reinforcing wires were anchored to the steel stanchions. The maximum lateral pressure carried by this wall was only about ¾ lb. per sq. in. Even this low value, however, represents an adequate margin of safety against wind.

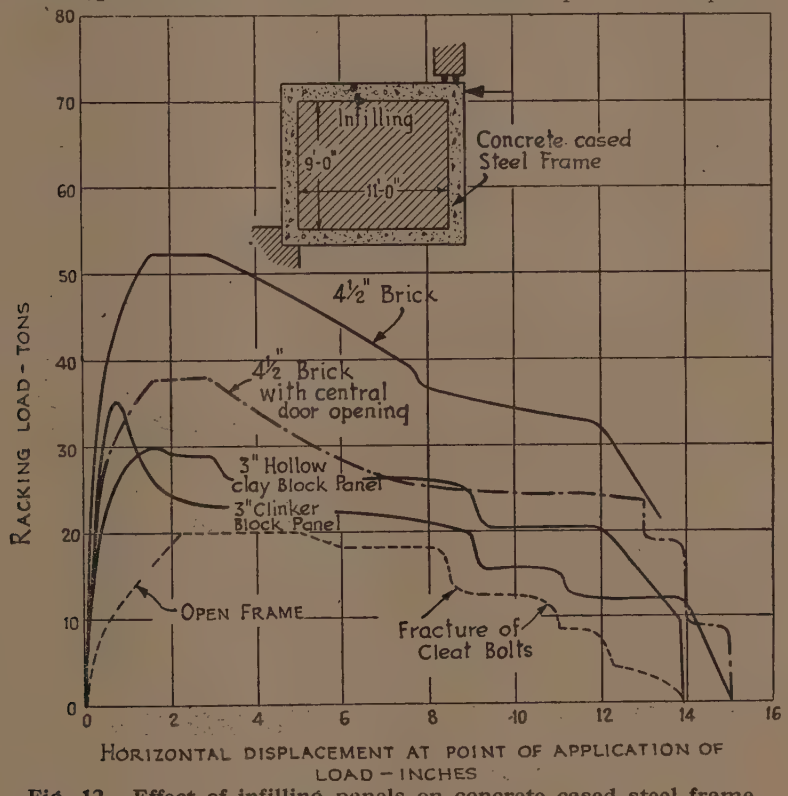


Fig. 12.—Effect of infilling panels on concrete cased steel frame subjected to racking load

Racking Strength of Panel Walls

In a framed structure the frames are subjected to a racking action as a result of wind pressure, but tests on houses showed that the cladding elements helped considerably in stiffening and strengthening the frame. At the request of the Ministry of Works, tests have been started to determine how important this help can be in large multi-storey buildings. A special testing-frame was designed by the Ministry and supplied as part of the equipment of a new Engineering Laboratory at the Building Research Station. This frame, shown in Fig. 11, is capable of taking a thrust of 200 tons at

selected positions. In testing a panel, a concrete block is interposed between a lower corner of the panel and the testing frame, and a horizontal thrust is applied to the diagonally opposite corner by means of a hydraulic jack. Two steel rollers are located at this loaded corner between the upper surface of the panel and a brick abutment clamped to the testing frame ; in this way tilting of the panel during test is avoided and horizontal displacement can occur without undue friction.

The results of some of the racking tests are shown in Fig. 12. In most of the tests the panel tested consisted of an encased steel frame similar to that used for the lateral loading tests, with an infilling of brickwork or other building blocks. In one test the encased steel frame was tested without any infilling. It will be observed from the figure that the stiffness of the frame was increased considerably even with a relatively weak infilling that might be used for a partition. The maximum racking load sustained with a 4½-inch brick infilling was over twice that which could be supported by the encased steel frame alone, and much of the help afforded by the brickwork was still retained when a door opening was left centrally in the panel. The form of the distortion of the brickwork in a racking test is shown in Fig. 13.

Fig. 13.—View of encased steel frame with 4½ inch brickwork infilling, after racking test

It is evident from these tests that allowance for the racking strength of walls and partitions might often avoid the present necessity for providing special connections or bracing in structural frameworks for resisting lateral forces.

Conclusions

(1) Analysis of the experimental data obtained on the strength of brickwork at the Building Research Station indicates that certain modifications are desirable in the design of brickwork, as at present recommended in Code of Practice CP111. These modifications include particularly :—

(a) Increased working stresses for brickwork with cement-lime mortars or with lime mortar, when high-strength bricks are used.

(b) Increased working stresses for slender piers and walls, when the eccentricity of loading is small.

With the modifications proposed, there would be a more reasonable distribution of the value of the load factor for a wide range of conditions (of type of brickwork, of slenderness, and of eccentricity of loading).

(2) Tests have shown that, although the tensile strength of brickwork is low and variable, considerable resistance to lateral forces can be developed by wall panels built into a steel framework. Such panels also have a considerable stiffening and strengthening effect on the resistance of the frame to racking forces.

Acknowledgements

The work described in this paper forms part of the programme of the Building Research Board of the Department of Scientific and Industrial Research and is published by permission of the Director of Building Research.

The tests on wall panels subject to lateral or racking loads were made at the request of the Ministry of Works and the results given in Figs. 10 and 12 are published by permission of the Director-General of Works in that Ministry.

Soil Mechanics in Relation to Structural Engineering*

By P. L. Capper, T.D., M.Sc., M.I.Struct.E., A.M.I.C.E. (Member of Council)

Synopsis

Most structural engineers realise that modern developents in Soil Mechanics can be of great value in the onomical and efficient design of structures of all nds. Theory and research have produced a mass of formation on this subject, much of it still controversial, ıt its full application to practical engineering problems ppears to lag behind. The object of this paper is to view the position of the science of Soil Mechanics in lation to certain aspects of structural engineering, to oint out some of the gaps in our present knowledge nd to indicate the lines on which design methods should e adapted so as to make the most efficient use of the vailable knowledge.

The applications of Soil Mechanics to structural ngineering cover a very wide range, and in this paper he author confines his attention to the more limited eld of the design of foundations. The important pplications of soil mechanics to construction problems le-watering, support of excavations, etc.), are outside he scope of this paper. The subject of pile foundations, so, is a very complex one, and is referred to only in a eneral way in relation to other types of foundation.

The subject is considered under the following headings:

Mechanics of Foundations. In order to apply theory nd the results of tests, the way in which a foundation pports its load and the factors governing its stability ust be thoroughly understood.

Site Investigation. The carrying out of a site investiation is now usually entrusted to specialists, who ossess the necessary equipment for boring, sampling nd testing ; but the structural engineer must be able decide on the extent of investigation necessary. He ust also be competent to appreciate the results of the vestigation and to apply them effectively in his esign.

Properties of Soils. The ability of soil to carry a perimposed load depends upon its shear strength and s compressibility. These properties vary, not only ith the type of soil, but also, for any particular soil, ith its water content. Recent research, correlated ith theory and with observations in the field, has done uch to clarify the factors governing the stability of undations as regards both complete collapse and ngerous settlement.

Design of Foundations. A knowledge of the way in hich the soil supports the foundation load, coupled ith adequate data regarding the soil properties, enables e designer to decide upon the most economical form d size of foundation.

Influence of Soil Mechanics on Design of Structures. A udy of the behaviour of soil under load shows that the gidity or flexibility of a structure has an important fluence upon the design of foundations. Accordingly, the selection, where practicable, of the type of structure most appropriate to the soil conditions may lead to considerable economy.

Future Development. A review of the subject reveals that there are still a number of problems on which more information is required. Research work is already in progress on some of these ; in other cases close cooperation between practising engineers and research workers will eventually provide solutions.

Paper to be read before the Institution of Structural Engineers 11, Upper Belgrave Street, London, S.W.1, on Thursday, bruary 26th, 1953, at 6 p.m.

Introduction

Up to the early part of the present century the design of foundations was based mainly upon empirical rules, intuition and experience. Knowledge of the properties of materials of construction led to the concept of a " safe bearing pressure " for any particular type of soil which would provide an adequate factor of safety against collapse and a negligible amount of settlement. The selection of this " safe bearing pressure " usually depended on classification of the soil by inspection. Engineering reference books, building bye-laws, etc., contained lists of soils with their bearing capacities. As there was no uniformity of classification or nomenclature the description of any particular soil varied according to the individual notions of the engineer or foreman who inspected it. Small-scale loading tests, often of a very rough-and-ready character, were used for more important work. The advocates of loading tests often ignored the fact that under a large bearing area the compression of the soil extends to a much greater depth than under a small area with the same intensity of loading.

The early theories of Coulomb and Rankine were directed principally to pressure on retaining walls, though it was Coulomb who first suggested the now well-known cohesive-frictional law which has become accepted as the basis of modern concepts of the failure of soils and of other materials. Rankine applied his conjugate pressure theory to the calculation of the safe depth of foundations on cohesionless soil, and in 1915 Bell extended this theory to cohesive soil. It is now generally recognised that as regards the stability of foundations the theories of Rankine and Bell are not supported by experimental evidence.

It was during the period between the two world wars that the science of Soil Mechanics as we now know it came into being. At first there was a tendency to place emphasis on simple classification tests : particle-size analysis, determination of liquid and plastic limits, and certain other tests which are now little used. It was hoped that by means of these classification tests any soil could be placed in a definite category, and that its engineering properties would then be established.

When comparing soils with other materials two essential difficulties arise, namely, the lack of homogeneity of natural soil, and the great variation of the properties of the soil with the water content. It may be largely because of these two uncertain factors that soil mechanics has for so long lagged behind other branches of engineering science.

Studies of a more fundamental character resulted in the evolution of theories for the distribution of stresses and strains in the soil beneath a loaded foundation, for the consolidation of compressible soil, and for the ultimate bearing capacity of soil. Laboratory and *in situ* tests were devised for the determination of the fundamental soil properties and empirical data necessary for the application of these theories to practical problems. Of recent years more and more attention has been paid to the correlation of theories and laboratory testing with practical observations in the field and with the analysis of failures.

Terzaghi[1], in his presidential address at the 2nd International Conference of Soil Mechanics and Foundation Engineering at Rotterdam, 1948, stated : " It has become more than evident that the essence of Soil Mechanics is a compromise between the status of an exact science, such as theory of structures, and that of an empirical one, like geology. The days when significant discoveries could be made in the laboratory or at the writing desk appear to be gone ; further progress depends chiefly on the improvements of methods of measurements on the field, on the scope and quality of field observations, and on the adaptation of the methods of subsoil exploration to practical requirements."

Many of the older engineers have been mistrustful of Soil Mechanics. Perhaps in the early days the enthusiasts tried to claim too much. It has been argued that practical experience is worth more than the work of the mathematician and the laboratory technician, and that the old empirical methods have always stood us in good stead. But have they ? Are not structural engineers constantly being asked to prepare plans for underpinning and other costly repairs to remedy defects in foundation design ? And what of the countless buildings which have collapsed and been demolished, and have been forgotten with the passage of time ? Nevertheless, the value of experience and common sense must not be underestimated. In fact, all soil mechanics specialists are agreed that without these qualities no engineer can hope to carry out his job successfully, however much information about the soil he may get from soil mechanics tests and theory.

Authorities on soil mechanics do not claim to provide a one-hundred-per-cent. correct answer to all our practical problems. However, as the result of all the work of the past 30 or 40 years it stands to reason that a considerable amount of valuable knowledge has been gained of the way in which the various factors affect the behaviour of foundations. Recent years have seen, on the one hand, increasing size and weight of structures, and on the other, increasing costs of labour and material, as well as shortages and restrictions of supply. Thus the problem of economical and efficient design of foundations is more than ever of paramount importance.

The Mechanics of Foundations

From the point of view of the way in which the load is transmitted to the soil most kinds of foundation, other than piles, may be divided into three classes :

Strip footing, such as the base of a wall, in which the load is spread only at right-angles to the wall ;

Pad footing, for example, that under a pier or column, in which the load is spread in all directions ;

Raft foundation, in which a number of walls or columns are supported on a system of continuous beams and slabs.

The transmission of the load from the structure to the soil may be considered in four stages : the contact pressure between footing and soil, the distribution of stress in the soil beneath the foundation, the deformations or settlements accompanying these stresses, and the form of failure caused by overloading.

Contact pressure. It is usually assumed that for symmetrical loading the pressure is distributed uniformly over the contact area. Theoretical analysis as well as experimental investigations show that this assumption is generally incorrect. For rigid footings on cohesive soil the pressure is greatest at the perimeter of the footing (Fig. 1 (a)), but local deformation of the soil under stress concentration tends to produce a more nearly uniform pressure distribution. On cohesionless soil the contact pressure is maximum at the centre (Fig. 1 (b) and (c)). When the footing is relatively flexible the pressure distribution is considerably modified. For eccentric loading on a footing it is customary to assume the conventional trapezoidal or triangular pressure distribution. This again is open to question, and Figs. 1 (d) and (e) show the general type of pressure distribution to be expected.

It can be shown that at depths below a footing greater than its width the contact pressure distribution assumed

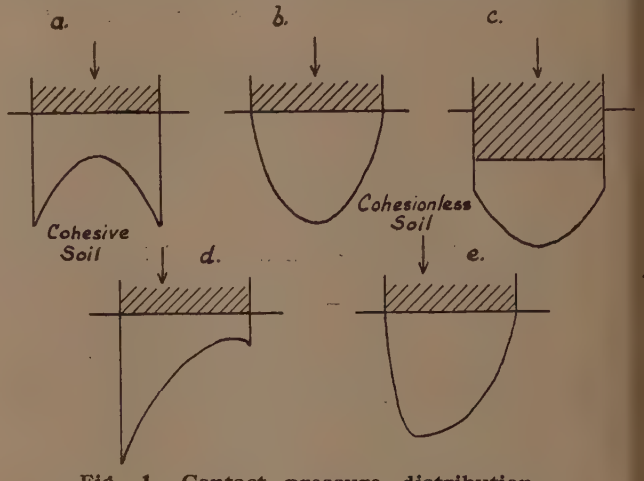


Fig. 1.—Contact pressure distribution

has very little effect upon the stresses in the soil, and consequently upon the settlement. The chief importance of contact pressure distribution lies in the determination of the bending moments and shears within the footing itself, particularly when the load imposed by a column, wall or pier has to be spread over a relatively large area of ground. For economical design the dimensions of the footing slab should depend on the contact pressure distribution, and this in turn depends on the flexibility of the footing.

Stress Distribution in Soil below Footing

The stresses in the soil beneath a footing are generally computed by Boussinesq's theory, which assumes a homogeneous elastic medium. The formulæ are cumbersome, but tables and diagrams of " influence factors " have been prepared[2], [3], by means of which the vertical pressures at any depth can easily be calculated. It may be asked " Why use a complex mathematical theory when the fundamental assumptions of an elastic and homogeneous material are far from true ? " The answer is that the mathematicians have done all the laborious work for us, and experiments with pressure cells have indicated that at any rate as regards vertical stresses, the calculated values are fairly near the truth, provided the soil is reasonably uniform. Why not, therefore, be grateful to the mathematicians and make use of the fruits of their labours ?

When there is considerable variation in the soil within the depth affected, the theory for homogeneous material is no longer applicable. Mathematical analyses have been made of the stresses and strains in elastic systems consisting of two or three layers having different elastic moduli. The theory, originated by Burmister in America, has been extended by the Mathematical Division of the National Physical Laboratory. Up to the present the theory has been developed mainly for pavement design,[4] but it should be possible to adapt it for the determination of the stress distribution in layers of soil underlying the foundation of a building.

Fig. 2 shows the theoretical distribution of vertical pressure in an elastic medium below a uniformly loaded circular area. The volume enclosed by a particular stress contour—for example that for which the vertical

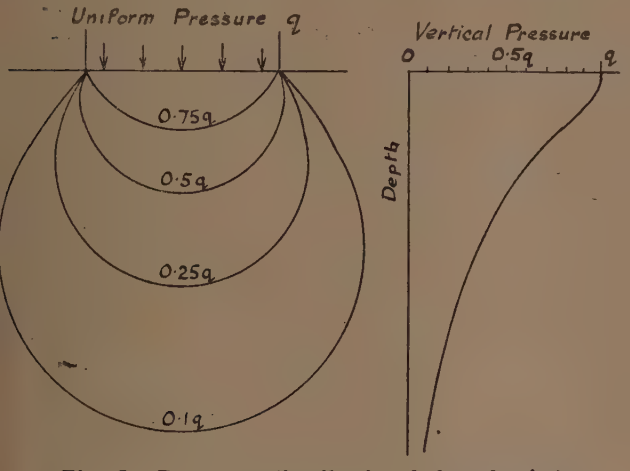


Fig. 2.—Pressure distribution below footing

pressure is 10 per cent. of the contact pressure—is known as the "pressure bulb." The pressure bulb may thus be defined as a zone beneath the footing in which appreciable stresses and deformations are set up by the load. This conventional idea is useful as a guide to the minimum depth to which site investigation should be made, as well as to the interaction which may be expected between foundations which are relatively close to one another. Fig. 3 shows some examples of pressure bulbs. The bulb under the large foundation (a) shows that the strength and compressibility of the soft stratum require special investigation. It is evident that the effects of such a stratum would not be revealed by the small-scale loading test represented at (b) in the diagram. The third example (c) shows the cumulative effect of closely-spaced loads.

Settlement. In general the settlement of a structure may be divided into the *immediate settlement* due to elastic and plastic deformation of the soil, and the *consolidation settlement* caused by the pressing out of pore water from the soil. In cohesionless soils where the permeability is high these two occur practically simultaneously; in cohesive soils consolidation is a relatively slow process. The above forms of settlement are the direct consequence of the load imposed by the structure upon the soil beneath. Settlement may, however, be induced or aggravated by other causes such as mining operations, changes in the water content of the soil, or disturbances due to nearby excavations, pile-driving or drainage.

Failure of foundations. Failure may occur in two ways. If the settlement, or more particularly the differential settlement between adjacent parts of the structure, becomes too large, the superstructure will develop cracks which is serious enough will constitute failure. Alternatively, collapse may take place on account of shear failure of the soil, accompanied by heaving up of the soil on one or both sides of the footing.

Site Investigation

The general principles and methods of site investigation are fully set out in the Code of Practice "Site Investigations,"[5] and a paper by Harding[6] gives much useful information about the methods and equipment required. The author therefore confines his remarks in this section to a brief summary of some of the more important points.

Before planning the site investigation in detail it is usually possible to obtain useful preliminary information from geological maps, from direct application to H.M.

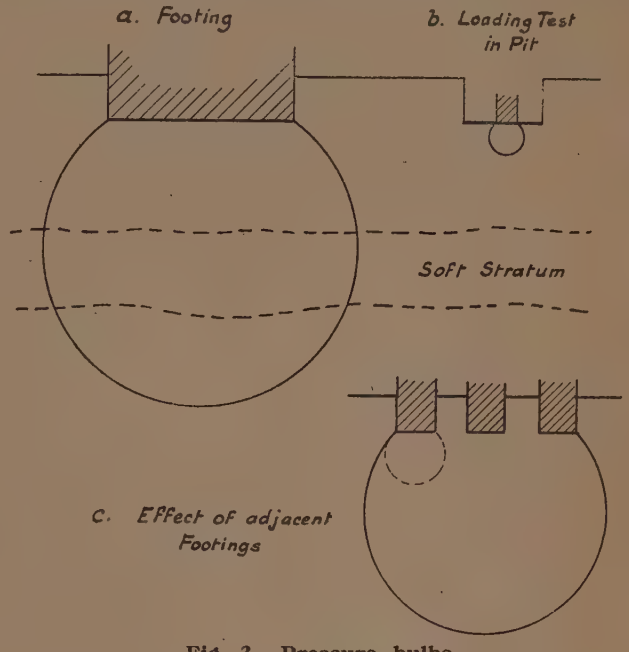


Fig. 3.—Pressure bulbs

Geological Survey, and from engineers and others who have been concerned with constructional work on neighbouring sites. Such information, in conjunction with the general character of the structure to be erected should enable the designer to decide on the extent and nature of the soil investigations needed.

As has already been mentioned, site investigation is generally entrusted to specialists. The planning of the investigation is best done by the Consulting Engineer in close co-operation with the engineer responsible for directing the work. Site investigations are often put out to tender, but it is probably more satisfactory in most instances to have the work done on a "cost plus" basis. Tendering for such work is very uncertain, and may almost amount to a gamble on the part of the contractor. He is faced with the problem of protecting himself against unexpected difficulties involving heavy costs, and yet keeping his price at a competitive level. It is most important that adequate time should be available for the carrying out of the site investigation in a thorough and systematic way. Too often the job has to be rushed, with unsatisfactory results.

A site investigation comprises the determination of the nature and properties of the soil at a sufficient number of points, distributed horizontally over the area of the site, and vertically through the maximum depth likely to be appreciably affected by the superimposed load. Where the strata are known to be fairly uniform, the number of test pits or boreholes can be kept small. If,

however, there is considerable variation over the site, even a large number of boreholes may not provide sufficient information, and the engineer must be prepared for unexpected variations and difficulties when construction commences. It may sometimes be desirable to carry out a small preliminary investigation, and if the results of this show that it is necessary, a more elaborate soil survey can be undertaken.

The procedure resolves itself into the following: access to the soil for sampling, extraction of samples, testing, and interpretation of the results.

For relatively shallow depths, down to about 15 ft., the most economical method of access is the excavation of trial pits. These have the obvious advantage of permitting visual examination of the soil, while the extraction of samples is comparatively easy. A useful auxiliary tool is the post-hole auger, which can be used for depths up to 20 ft. in cohesive soil free from stones. For some small jobs the auger is adequate without the need for more elaborate equipment.

For most of the classification and identification tests the samples of soil may be in the disturbed state, and the extraction of such samples either from an excavation or from the bottom of a borehole presents no special difficulties. For shear and consolidation tests on cohesive soil, however, undisturbed samples are essential. The usual method is to drive a cylindrical sampling tube with a cutting edge into the soil, give it a twist to shear off the bottom of the sample, and then withdraw the tube. The ends are sealed up to preserve the natural moisture content.

The extraction of undisturbed samples of cohesionless soil has hitherto presented great difficulties. A device for this purpose has been developed by Bishop[7] and it is claimed that from the samples thus obtained the properties of the soil in its undisturbed state can be measured with fair accuracy.

The following tests are usually required:

Tests	Remarks
Classification and Identification Tests	
Natural water content	
Specific gravity of soil particles	Necessary for wet analysis.
Particle-size analysis	Sieving for coarse material; wet analysis for silt and clay fractions.
Liquid and plastic limits	Fine-grained soils only.
Soil Properties	
Density	Undisturbed and sand replacement methods.
Shearing Strength	Shear box, triaxial and unconfined compression, vane.
Compressibility	Oedometer test.

Of the above, all the tests except those for shear strength and compressibility are specified in B.S. 1377 (1948) "Methods of Test for Soil Classification and Compaction."[8]

From the visual inspection and the results of the classification tests the various types of soil encountered on the site can be classified in accordance with Table I in the Code of Practice, "Site Investigation." In this classification system soils are divided into three main groups: cohesionless, cohesive and organic. The soils in these groups are further sub-divided according to predominant particle-size and physical characteristics, and certain composite soil types are included.

In addition to the information necessary for the classification of the soil according to its composition and physical properties, certain geological data should always be recorded. This should include the geological origins of the soil deposits (glacial, alluvial, wind-borne, etc.), and their relation to the well-known geological systems and series. A sound appreciation of the geological conditions of the site can do much to obviate serious mistakes which sometimes arise from borehole observations. Familiar examples include boulders mistaken for bedrock, small pockets of sand or gravel assumed to be a main stratum, and the failure to detect a fault or irregularity in the geological formation.

A simpler classification is given in the Table I (Bearing Capacities) in the Code of Practice "Foundations."[9] Here nineteen types of "soil" are specified, arranged in five groups: rocks, non-cohesive soils, cohesive soils, peat and made ground.

On account of the difficulty of extracting undisturbed samples of sandy soil, and because the strength of such soil varies considerably with the degree of compaction, penetration tests are often used to determine the properties of the soil *in situ*. In the American standard penetration test described by Terzaghi and Peck,[10] a sampling spoon of standard design is driven into the soil at the bottom of a borehole by blows from a drop weight. From the number of blows required to produce a specified penetration the density and compressibility of the soil can be estimated, and the samples brought up by the spoon are used for identification and classification purposes.

Other forms of *in situ* penetration test have been developed, principally for the measurement of the bearing capacity of soil for pile foundations. Examples of these are the Dutch "deep-sounding" procedure[11] and the "pre-piling" method described by Gower Pimm,[12] in which the toe resistance and the skin friction can be separated.

Soil Properties

The principal soil properties which affect foundation design are shearing strength and compressibility.

The shearing strength of soil is usually considered to consist of cohesion, internal friction or a combination of these, and may be represented by Coulomb's equation:

$$s = c + \sigma \tan \varnothing,$$

where s = shearing resistance,
c = cohesion at zero normal stress,
σ = normal stress on shear plane,
$\varnothing$ = angle of shearing resistance.

Strictly speaking, the term compressibility denotes the contraction in volume under the action of pressure, this contraction being due to reduction of voids by the expulsion of water. The rate at which this contraction takes place depends on the permeability of the soil and the length of the drainage path by which the water escapes. The settlement of a structure depends not only on the compressibility of the soil as defined above but also on the elastic and plastic deformation of the soil structure.

Shearing Strength

Shear Tests. The principal laboratory methods of measuring the shearing strength of soil are the shear box, the triaxial compression and the unconfined compression.[13] In the first a specimen of soil is sheared directly by a horizontal force, while a constant normal pressure is maintained on the plane of shear by means of a dead load. The results of the test are plotted

rectly as in Fig. 4 and the constants c and ϕ are read from the graph.

In the triaxial apparatus a cylindrical sample enclosed a thin rubber membrane is subjected to uniform ter pressure. The axial load is then steadily in-ased until failure takes place. If σ_3 is the uniform eral pressure and σ_1 the axial compressive stress at

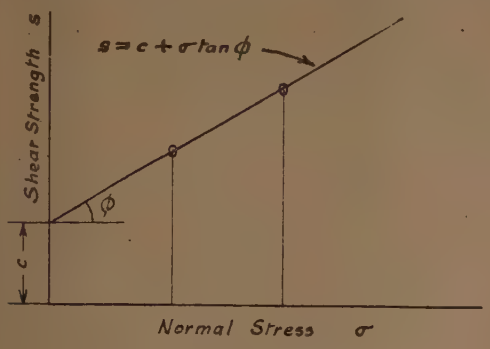


Fig. 4.—Coulomb's Law

ilure, the stress conditions may be represented by the ohr stress circle (1) as shown in Fig. 5. If the hori-ntal axis of the diagram represents the plane on which e major principal stress σ_1 acts, then the line AR presents the plane of rupture. The coordinates of R e respectively the normal stress σ and the shear ress s on that plane. Thus the application of the teral pressure is a means of applying a normal stress the plane of rupture. The test is then repeated ith a different lateral pressure, giving the circle (2). he constants c and ϕ are determined by drawing a mmon tangent (known as the Mohr envelope) to the o circles. In practice at least three tests are made order to define the Mohr envelope more accurately.

A simpler form of shearing test commonly used for turated clays is the unconfined compression test, which usually carried out on the site using portable appara-s. In principle it is a special case of the triaxial test which the lateral pressure σ_3 is zero. As only one ohr circle is obtained from this test the value of ϕ nnot be determined. The test is therefore restricted immediate or undrained tests on saturated clays for

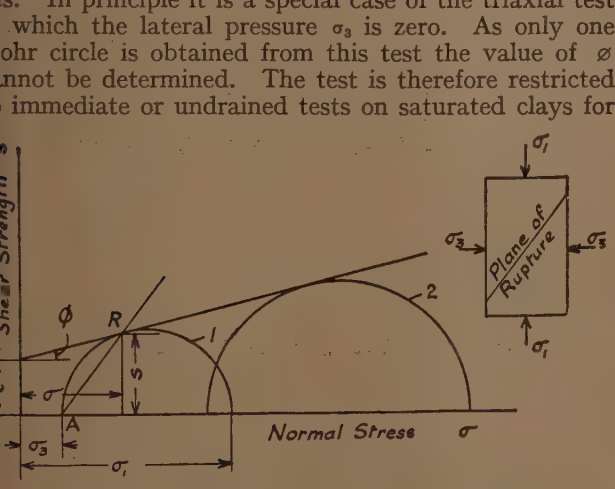


Fig. 5.—Shear strength from triaxial test

hich, as explained below, the angle of shearing resist-nce ϕ is zero.

Another form of direct shear test, known as the vane st,[14] has recently been developed. A cruciform vane pressed into the soil at the bottom of a borehole. easurement is made of the torque required to rotate e vane at a steady rate. It is assumed that the clay ears along the surface of a cylinder of diameter and ight equal to those of the vane. This test has advan-ges over other types for very soft clays, and it is aimed that it gives more reliable results at great depths where the extraction of representative undisturbed samples presents difficulty.

Coulomb Law. For an ideal material possessing both cohesion and internal friction and with physical properties which remain constant within ordinary ranges of stress the quantities c and ϕ would be constants, which could be determined by carrying out two or more tests with different normal pressures acting on the plane of shear failure. For soil, however, the quantities c and ϕ vary with its water content and density, and thus depend not only upon the state of the soil before loading but also upon the drainage conditions which apply during the process of loading and shearing. When a pressure is applied to saturated soil it is resisted partly by the inter-granular pressure σ_e and partly by the water pressure u in the pore spaces. The pressures are related by the equation

$$\sigma = \sigma_e + u$$

The frictional resistance depends on the intergranular pressure, which for this reason is often termed the "effective" pressure. Fundamentally the shear strength equation is of the form

$$s = c + \sigma_e \tan \phi = c + (\sigma - u) \tan \phi$$

but since the pore-water pressure u is often unknown it is more convenient to use the applied pressure σ instead of the effective pressure. The equation therefore becomes

$$s = c + \sigma \tan \phi$$

in which the quantities c and ϕ are considered as empirical constants rather than fundamental soil properties. For this reason c is generally called the *apparent* cohesion and ϕ the *angle of shearing resistance.*

Drainage conditions. Shearing tests for soils are classified as follows :—

(a) *Immediate or Undrained Tests* : the samples are subjected to applied pressure under conditions in which drainage is prevented. The samples are then sheared, also under conditions of no drainage.

(b) *Consolidated—Undrained Tests* : before testing, the samples are allowed to consolidate under a certain pressure ; they are then subjected to undrained tests as in (a).

(c) *Drained Test* : shearing is carried out slowly, simultaneously with consolidation, under conditions of no excess pressure in the pore space.

The terms "quick," "consolidated-quick" and "slow" are sometimes used to describe these three types of test, and suffixes u, cu and d are used to distinguish the coefficients c and ϕ for the above three conditions. The true cohesion and angle of internal friction are distinguished where necessary by the symbols c_e and ϕ_f respectively.

The choice of test conditions depends on the purpose for which the shear strength is required. The guiding principle is that the drainage conditions of the test should conform as closely as possible to those under which the soil will be stressed in practice.

Immediate or undrained tests are generally used for foundations on clay soils, since during the period of construction only a small amount of consolidation will have taken place, and consequently the water content will have undergone little change. With sandy soils consolidation occurs relatively rapidly and is usually completed during the application of the load. Tests on sand are therefore generally carried out under drained conditions.

Shear strength of sand. Sandy soils are generally regarded as being cohesionless, their shear strength being wholly dependent upon internal friction. The angle of friction ϕ_f is not constant, but varies with the

density of packing, being least when the sand is in the loose state. The *angle of repose*, that is, the angle to the horizontal at which a heap of sand will stand without support, is approximately the same as the angle of friction in the loose state.

Shear strength of clays. With cohesive soils the values of the constants c and $\varnothing$ vary with the drainage conditions. The results of immediate or undrained shear tests (Fig. 6) on saturated clay show that the shearing resistance is apparently independent of the normal pressure. In other words, the angle of shearing resistance $\varnothing_u = 0$. This is accounted for by the fact that when the normal pressure is applied it is at first taken up by the pore water without increasing the effective pressure. It is often erroneously inferred that clays possess no internal friction. If the angle of internal friction were zero the plane of shear failure in a compressive test would be at 45° to the cross-section of the specimen. In such tests the inclination of the plane of failure to the cross-section is always greater than 45° showing that internal friction does exist.

The assumption that under undrained conditions the angle of shearing resistance $\varnothing_u$ (as distinct from the true angle of friction) is zero is often known as the "$\varnothing = 0$" assumption.[15] It leads to considerable simplification of stability analysis, and appears to be justified by the results of practical observations under conditions which correspond closely to those of the undrained test. Where this assumption is valid the unconfined compression

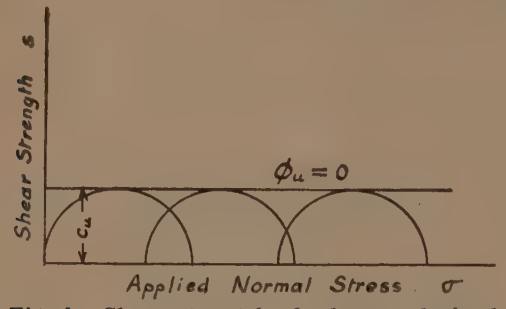


Fig. 6.—Shear strength of clay—undrained

test or the vane test can be used to determine the apparent cohesion c_u.

If the applied pressure continues to act for some time the soil is consolidated, its void ratio is reduced, and its true cohesion is increased. If the true angle of friction, $\varnothing_t$, were known, a Coulomb line could be drawn for any given void ratio. In Fig. 7 two such lines are shown, corresponding to two different values of the void ratio. The lines are not necessarily quite parallel, as $\varnothing_t$ may vary slightly with the void ratio.

The consolidated-undrained test is carried out in two stages. During the first stage drainage, and therefore consolidation, takes place under a certain pressure p. During this consolidation the void ratio of the soil is reduced from, say, e_1 to e_2, with a corresponding increase in cohesion. If several samples which have been consolidated under the same pressure p are then tested with different normal pressures on the plane of shear the results will be similar to those of the immediate undrained test. Thus for saturated clay $\varnothing_{cu} = 0$, but the apparent cohesion c_{cu} has a value corresponding to the void ratio after consolidation.

In the drained test, the consolidation and shearing processes take place simultaneously, but at a sufficiently slow rate for changes of water content to keep pace with changes of stress. The effective stresses are equal to the total stresses. The Mohr circle for each test is tangent to the Coulumb line corresponding to the void ratio at which failure takes place. The envelope of these circles is the line AB (Fig. 7) from which the constants for the drained test c_d and $\varnothing_d$ are obtained. It will be noticed that $\varnothing_d$ is greater than $\varnothing_t$.

Partially saturated cohesive soils, as well as some saturated silts and some mixtures of sand and clay exhibit an angle of shearing resistance (undrained) $\varnothing_u$ greater than zero. Such soils are sometimes termed cohesive-frictional or "c-$\varnothing$" soils.

For stability analysis it is often convenient to consider soils as falling into one of the following idealised types:

(a) Frictional ($c = 0$)
(b) Cohesive-frictional (c-$\varnothing$)
(c) Cohesive ($\varnothing = 0$)

Variation of Shear Strength of Cohesive Soil with Depth. In a uniform deposit of clay, normally consolidated under its own weight, the void ratio decreases with the

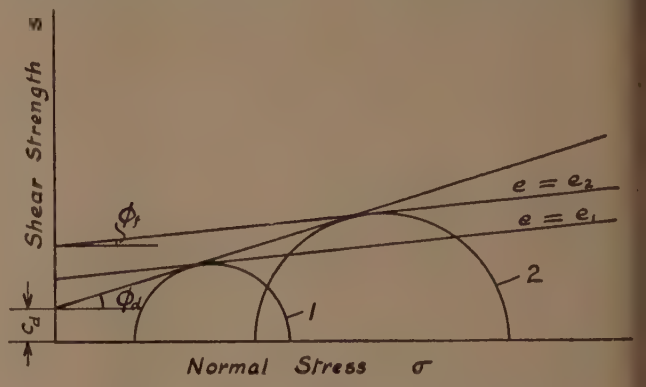


Fig. 7.—Shear strength of clay—drained

depth. Consequently the strength increases with depth as shown by the line OB in Fig. 8, and the ratio of the shear strength c to the overburden pressure p is approximately constant. It has been found, however, that, particularly with sensitive clays, the ratio of the actual shear strength to the overburden pressure is appreciably less than that estimated from samples consolidated in the laboratory. The discrepancy is probably due to the lateral pressures in the ground being less in relation

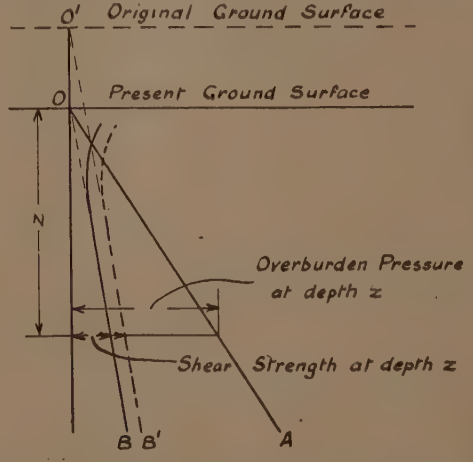


Fig. 8.—Variation of shear strength with depth

to the vertical pressures than in the laboratory consolidation process.

With over consolidated clay the shear strength is roughly proportional to the depth below the level of the ground at the period when consolidation originally took place, as shown by the line $O'B'$. Near the surface the strength is affected by partial drying out of the clay and the shear strength curve may at first show a decrease with depth.

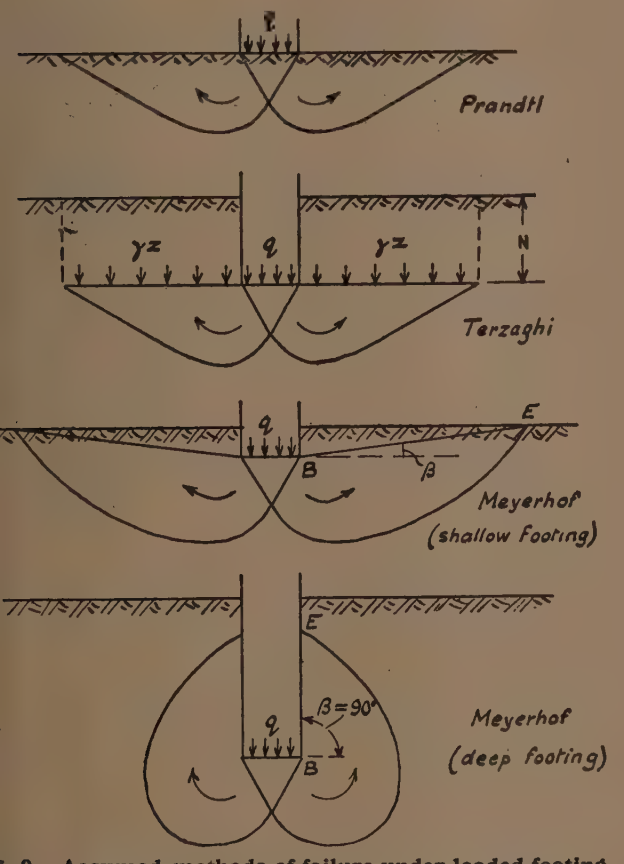
It is well established that the shear strength of remoulded clay is generally less than that of the same soil

the undisturbed state. The loss of strength varies y much with different types of clay. Clays which w a considerable loss of strength are known as ensitive." Sensitivity is often of great practical portance when clay soil is disturbed by pile-driving, avation or other causes.

TIMATE BEARING CAPACITY

The *ultimate bearing capacity* of a soil is the contact ssure between the footing and the soil at which shear ure of the form shown in Fig. 9 takes place. Most ories for ultimate bearing capacity are based on the haviour of an ideal material. Modifications are made simplify the analysis, to allow for the frictional perties of the soil, and to bring the theoretical and served forms of failure into closer agreement. Skemp-n[16] gives a critical review of various theories, as applied cohesive soil, and correlates them with experimental ta.

Fig. 9 shows the mechanism of failure on which some these theories are based. Prandtl (Fig. 9 (a)) con-



g. 9.—Assumed methods of failure under loaded footing

ered a load applied to the surface of a plastic medium, e weight of the material displaced being neglected. r cohesive soil ($\varnothing = 0$) the ultimate bearing capacity of a strip footing is given by

$$q_u = (\pi + 2)\, c = 5.14c.$$

Later theories, such as that of Terzaghi (Fig. 9 (b)) ke into account the weight of the displaced soil, and footings below the surface the effect of the over-rden is treated as a surcharge load applied at the el of the base. Meyerhof[17] has carried the analysis a ge further as shown in Figs. 9 (c) and 9 (d), which dicate the assumed soil displacements for shallow and ep footings respectively.

The generally accepted formulæ for ultimate bearing capacity are of the form :

$q_u = cN_c + \gamma z N_q + 0.5\, B\gamma\, N_\gamma$ for a strip loading

$q_u = 1.3\, c\, N_c + \gamma z N_q + 0.4\, B\gamma N_\gamma$ for square footings

$q_u = 1.3\, c\, N_c + \gamma z N_q + 0.3 B\gamma N_\gamma$ for circular footings

where

q_u = ultimate bearing capacity
c = cohesion
γ = density
B = breadth of footing
z = depth of footing
$\varnothing$ = angle of shearing resistance
N_c, N_q, N_γ = bearing capacity coefficients.

The first term in these equations represents the effect of cohesion, the second the effect of depth, and the third that of breadth in increasing the ultimate bearing capacity.

Curves showing Terzaghi's values of the bearing capacity coefficients for different values of $\varnothing$ are given in most text-books on Soil Mechanics, and these are suitable for relatively shallow foundations. Meyerhof, from his theory of deep foundations[17], has deduced a series of curves showing the variation of the coefficient with depth as well as with the angle of shearing resistance. The depth parameter is expressed by the inclination β of the "equivalent free surface"—the line BE in Figs. 9 (c) and (d). For the deep footing in Fig. 9 (d) it will be seen that $\beta = 90°$. Some examples of Meyerhof's curves giving the bearing capacity coefficients for strip loading are shown in Fig. 10.

When the base of the foundation is below ground level it is usual, when finding the maximum safe bearing capacity, to consider the nett pressure on the soil, that is, the pressure additional to the overburden pressure. Thus with a load factor F the maximum safe bearing capacity q_o for a strip footing would be given by

$$q_o = \frac{1}{F}\left\{cN_c + \gamma z\,(Nq - 1) + 0.5\, B\gamma\, N_\gamma\right\} + \gamma z$$

Bearing Capacity of Cohesive Soil. For cohesive soils under undrained conditions $\varnothing = 0$, and $N_q = 1$ and $N_\gamma = 0$. The equation for strip footings thus becomes

$$q_u = c\, N_c + \gamma z$$

in which the second term represents the pressure of the overburden removed.

The following simple rules have been suggested by Skempton[16] for the ultimate bearing capacity of cohesive soil :

Depth	N_c
At surface	5 for strip footing. 6 for square or circular footing.
$D/B < 2\frac{1}{2}$	$(1 + 0.2\,\frac{D}{B}) \times N_c$ at surface.
$D/B > 2\frac{1}{2}$	$1.5 \times N_c$ at surface.
At any depth	N_c for rectangle ($L \times B$). $= \left\{1 + 0.2\,\frac{B}{L}\right\} \times N_c$ for strip.

It can be shown that when a strip load of width B and of intensity q per unit area is applied to the surface of an elastic medium, the maximum shearing stress occurs at a point vertically below the centre of the load at a depth $B/2$, and its value is q/π. Thus in cohesive soil of shear strength c the shear stress at that point will reach its ultimate value when $q = \pi c$ or $3.14c$. Complete failure does not occur until q reaches a value of more than $5c$. Between these values of q, since the overstressed soil is laterally confined, redistribution of stress takes place with a limited amount of plastic flow. In soft soils, however, it is possible for progressive settlement from this cause to occur before the condition for complete failure is reached. This is known as "local shear failure." By using an adequate load factor against complete failure, and by limiting the allowable settlement as explained below, local shear failure can usually be disregarded for shallow foundations. For very deep foundations local shear failure is more likely to be of importance, and has been taken into account by Meyerhof in evaluating his bearing capacity coefficients.

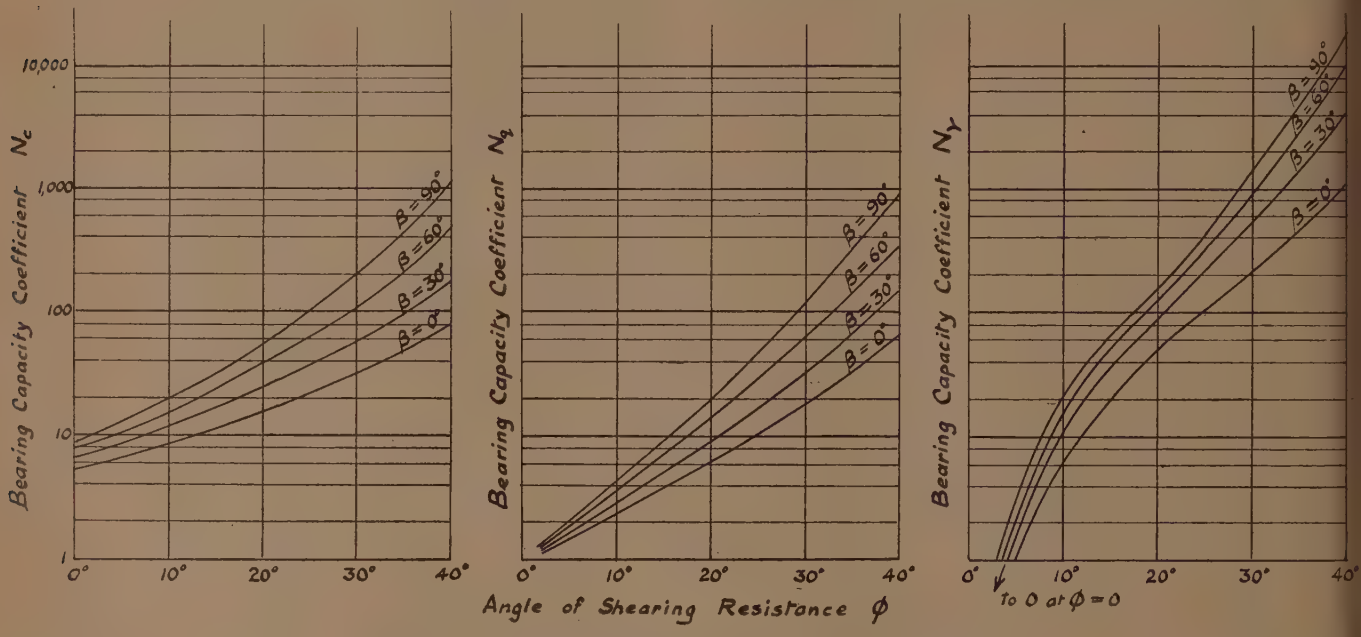


Fig. 10.—Bearing capacity coefficients (after Meyerhof)

Bearing Capacity of Cohesionless Soils. Since there is no cohesion, the first term in Terzaghi's expression for ultimate bearing capacity is zero. As the angle of friction ϕ increases the bearing capacity coefficients N_q and N_γ increase rapidly. The ultimate bearing capacity therefore increases indefinitely with the width and depth of the foundation, but as the settlement under a given intensity of loading tends to increase with the width, the allowable bearing pressure is governed by settlement considerations for all except relatively small or narrow shallow footings.

The bearing capacity of cohesionless soil is considerably reduced by saturation with ground water, since if the base of the foundation is below ground water level the effective density of the soil is reduced to its submerged value.

Compressibility and Settlement

When a compressive load is applied to the soil beneath a foundation, the deformation may be divided into :

(a) elastic deformation, which disappears when the load is removed ;

(b) plastic or non-elastic deformation ;

(c) consolidation or compaction by the close packing of the particles accompanied by the removal of water and air from the voids.

In fine-grained soils of low permeability the latter effect is very slow and may be considered separately from the others. The first two are often described as "immediate" settlement. Plastic flow is, of course, progressive with time, and the term "immediate" is used only in a relative sense. Fig. 11 shows a typical load/settlement curve for cohesive soil, in which nearly all the deformation is "immediate." By drawing a tangent to the curve at the origin the elastic and plastic components of the settlement can be separated.

Settlement of Cohesive Soil. The assumptions made in the elastic theory—homogeneous material and linear stress/strain relation—are not strictly applicable to soils. While it is generally accepted that provided the soil is fairly uniform the elastic theory can be applied without serious error to the determination of the vertical pressure beneath a loaded area, it is of doubtful value for estimating settlement. It can be used, however, to give a rough approximation to the immediate settlement of cohesive soil.

If a load is applied to the surface of an elastic medium the settlement ρ is given by an expression of the form

$$\frac{\rho}{B} = \frac{\alpha q}{E}$$

where B = width of loaded area
q = average intensity of loading per unit area
E = modulus of elasticity
α = a coefficient depending on the shape of the loaded area, on the distribution of contact pressure, and on Poisson's ratio for the material.

Even if this expression is accepted, the determination of a suitable value of E presents difficulties. The stress/strain curve for soil is not linear, hence E varies with the stress. The deformation produced by a loaded area is the summation of the deformations of a series of elements stressed to different amounts and therefore having different values of the modulus. Provided the

is reasonably homogeneous to a depth of at least ce the width of the loaded area, the value of α/E ermined from a loading test carried to the same nsity of contact pressure can reasonably be used.

kempton[16] compares the load/settlement curve ained from a bearing test on cohesive soil with the ss/strain curve from an unconfined or triaxial com- ssion test and points out the similarity in shape over

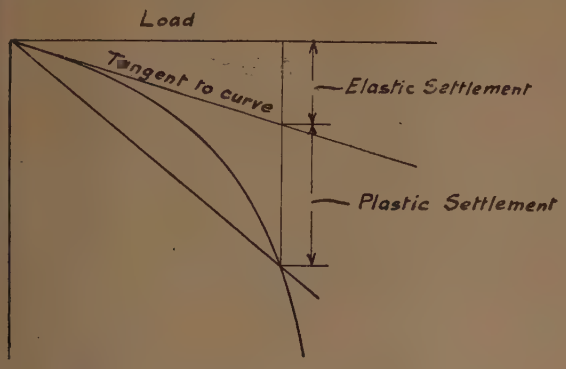


Fig. 11.—Typical load/settlement curve

ges of stress up to well over half the ultimate values. is similarity would appear to be coincidental, since a compression test the stress is approximately uni- m, whereas under the loaded area the stress distribu- n is very complex. Also, in the compression test the eral pressure is constant ; in the ground the lateral ssure on any element increases in proportion to the plied load. It can be shown that for a rigid circular ting on the surface of an elastic medium for which sson's ratio is 0.5 the value of α is approximately 0.6. empton therefore suggests that the equation

$$\frac{\rho}{B} = \frac{0.6q}{E}$$

be used to estimate the immediate settlement, E ng taken as the secant modulus obtained from an confined or triaxial compression test over an appro- ate range of stress.

Another method of predicting "immediate" settle- nt from loading tests is based on Housel's "perimeter- ar" theory. The bearing pressure q required to duce a specified settlement is given by the expression

$$q = \frac{P}{A} n + m$$

which P is the perimeter and A the area of the applied d, and m and n are empirical constants for the soil, ich can be found by making tests on bearing plates different sizes. The concept of perimeter shear is rely empirical, but with cohesive soil it appears to e results in accordance with experience. For co- sionless soil the "perimeter-shear" effect—the first m in the above equation—is very small.

The compressibility or consolidation settlement of esive soil is effectively demonstrated by the œdometer t. A laterally confined sample of soil is compressed tween two porous stone discs, which allow water to be ueezed out gradually. An increment of pressure plied to the soil is at first resisted by the pore water. der this head the water begins to flow towards th surfaces and to escape through the porous stones. cause of the low permeability of the soil the rate of flow is very slow. Eventually equilibrium is reached, and no further consolidation takes place. The volume of the sample has been reduced by that of the water expelled. The intergranular pressure is now equal to the applied pressure and the pore water pressure is zero. If the applied pressure is then reduced negative pore pressure (in other words tension) is set up, and water is sucked in causing an increase in volume. Since the soil matrix is assumed incompressible the volumetric change is almost entirely due to change in water content.

If a series of pressure increments is applied, each pressure being maintained steady for sufficient time for equilibrium to be reached, the curve of void ratio plotted against pressure thus obtained is of exponential form as shown by AB in Fig. 12. If when some point C is reached, the pressure is released by similar stages an expansion curve such as CD is obtained, much flatter than the compression curve. If the soil is afterwards re-compressed the new compression curve will follow the expansion curve closely as shown by the dotted line DC, and if compression is continued beyond C the curve will be a continuation of the original compression curve.

Soil deposits in nature have undergone a natural process of consolidation. Starting at the liquid limit when deposited in water, the soil has been subjected to increasing pressure as more and more material has been deposited above it. Water is gradually squeezed out and the soil consolidated.

In the course of geological history some deposits of soil have been consolidated and subsequently relieved of some of the pressure by erosion. Clays which have undergone steady natural consolidation are often described as "normally consolidated," while deposits which have been consolidated and then relieved of some

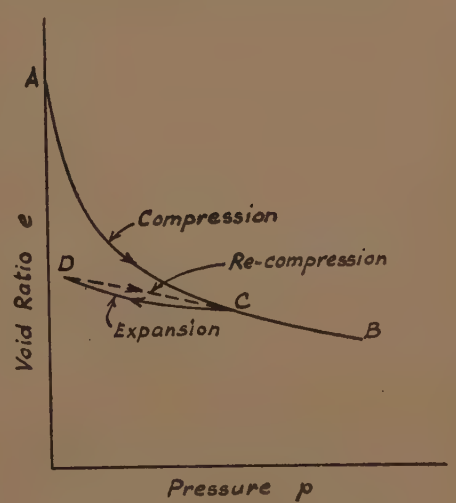


Fig. 12.—Void ratio/pressure curves

of the pressure are known as "over-consolidated." Soils of the latter type are naturally less compressible than normally consolidated soils.

The experimental pressure/void ratio curve often gives useful information about the past geological history of the deposits. It will be seen that in the process of natural consolidation for any pressure there is a corresponding void ratio for equilibrium. If, however, the soil has undergone compression followed by expansion and perhaps recompression, the equilibrium value of the void ratio will be less than that for the same pressure under conditions of original consolidation.

The probable settlement of cohesive soil due to con- solidation is usually estimated by means of Terzaghi's well-known theory. The factors involved are the depth

extent and properties of the compressible soil stratum, and the magnitude and distribution of the additional pressure imposed by the weight of the structure.

The principal assumption made is that drainage takes place in the vertical direction only. Drainage may be either two-way or one-way, according to whether permeable strata exist both above and below the compressible soil, or at only one of the boundaries.

In applying the results of the consolidation test to settlement analysis two coefficients are used.

The *coefficient of volume compressibility*, m_v, is the volumetric change per unit pressure increment. In symbols, for a sample of thickness h reduced an amount δh by an increment of pressure δp,

$$m_v = \frac{\text{change in unit volume}}{\text{change of pressure}} = \frac{\delta h}{h} / \delta p$$

which in the limit becomes $-\frac{1}{h}\frac{dh}{dp}$. Instead of calculating void ratios and plotting the pressure/void ratio curve, the thickness of the specimen h may be plotted against the pressure p. The slope of this curve divided by h is the coefficient m_v. This coefficient is the inverse of a pressure, and is usually expressed in the units ft^2/ton.

The *coefficient of consolidation*, c_v, which is used in estimating rate of settlement, is defined as the ratio $k/m_v\gamma_w$, where k is the permeability of the soil and γ_w the density of water. The coefficient of consolidation has the dimensions (length)^2/time, and is expressed in units such as in^2/min or ft^2/day.

The theoretical settlement/time curve is of the form shown in Fig. 13. Here, as in the œdometer test, the full load is applied immediately. For practical structures it is generally assumed that the lower part of the curve is approximately the same as if the full load were applied instantaneously at about the middle of the period of construction. It will be seen that, although the settlement may take many years to reach a state of equilibrium, a small but appreciable proportion of the final settlement, perhaps of the order of five per cent., may develop during construction.

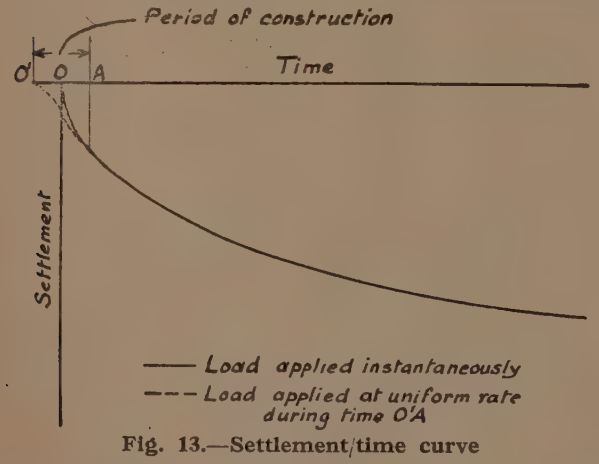


Fig. 13.—Settlement/time curve

The first step in settlement analysis is to calculate the pressure distribution through the compressible soil. Examples of pressure distribution curves are shown in Fig. 14. In one example the compressible stratum extends from immediately under the foundation to a considerable depth below. In the other there is a compressible layer underlying a stratum of sandy soil which is permeable and relatively incompressible. In either case the probable settlement ρ is given by

$$\rho = m_v A$$

where A is the area of the applied pressure diagram shown shaded in Fig. 14.

In order to allow for variations in the value of m_v at different depths the compressible soil may be divided into layers, and the settlements of these layers calculated and totalled.

When a structure is founded directly upon compressible cohesive soil extending to a considerable

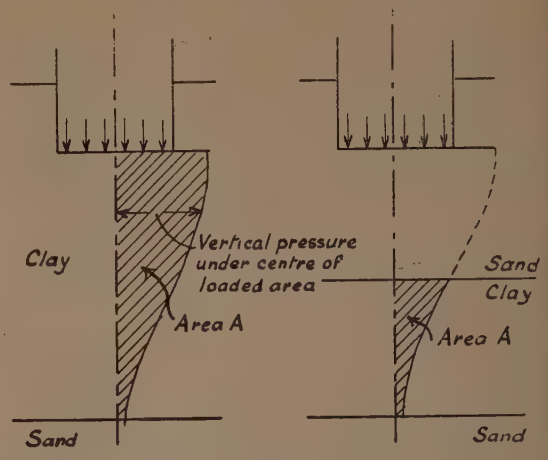


Fig. 14.—Pressure distribution under footing

depth, with a relatively permeable stratum underneath, a rough approximation to the settlement can be obtained as follows. It can be shown from the consolidation theory that for a square or circular loaded area the approximate settlement is given by

$$\frac{\rho}{B} = 0.8\, m_v q$$

where m_v is the average value of the coefficient of volume compressibility over the appropriate range of pressure as obtained from the œdometer test. It will be noted that the equation is similar in form to the expression for immediate settlement, since m_v is dimensionally equivalent to the reciprocal of an elastic constant.

In the preceding paragraphs we have been considering the mean settlement of a uniformly loaded area. It is well-known that a uniformly distributed load produces a dish-shaped settlement, for example, with uniform pressure on a square area the settlement at the corner is theoretically half that at the centre.

The shape of the settlement profile, as well as the distribution of contact pressure, is therefore dependent on the rigidity of the base. Thus, on cohesive soil conditions may range between a rigid footing, settling uniformly but with extremely high contact pressure at the edges (as in Fig. 1), and a flexible footing which conforms to the natural shape of the deformation caused by uniform pressure.

Settlement of Cohesionless Soil. Unfortunately the settlement characteristics of sandy soils are difficult to determine experimentally. They depend upon the angle of friction, which in turn depends upon the density or packing of the grains. Because of the difficulty of extracting undisturbed samples of sandy soil, a reliable estimation of settlement cannot be made from laboratory tests. Rough approximations can, however, be made from *in-situ* plate-bearing or penetration tests.

Terzaghi and Peck[18] give an empirical relation between the settlement ρ for a loaded area of width B ft. and the

tlement ρ_1 for a loaded area 1 ft. square under the ne intensity of loading q per sq. ft. of area

$$\frac{\rho}{\rho_1} = \left\{\frac{2B}{B+1}\right\}^2$$

This ratio tends towards the limit 4 as B becomes very ge.

For a specified maximum settlement ρ of the full-sized undation the settlement ρ_1 of the 1 ft. square plate is ven by

$$\rho_1 = \rho \left\{\frac{B+1}{2B}\right\}^2$$

The intensity of loading q_u required to give this settlent is the allowable bearing pressure for the settlement specified for the full-sized foundation.

Tests using a bearing plate of 1 sq. ft. area loaded , say, 4 or 5 tons/ft.^{2} are not easy to carry out, and the e of smaller plates is ruled out by the probability of ear failure. For this reason penetration tests such the Dutch deep-sounding test[11] and the American andard penetration test [10] are sometimes used to give rough guide to the allowable bearing pressure for a ecified amount of settlement. The results of such enetration tests must of course be correlated to observaons from bearing tests or to the measured settlements actual structures.

Design of Foundations

In designing the foundations with an adequate margin safety as regards the two types of failure, excessive ttlement and collapse, it is necessary first to consider e general design of the superstructure. Is it to be of exible or rigid construction? Is it to be an elaborate d relatively expensive building, or a utilitarian ructure to be erected at the minimum of cost consistent ith functional requirements?

The depth of the foundations will often be determined y considerations such as basement accommodation quired or levels of sewers, which must be taken into count. On compressible soil the benefits of deep sements in limiting the settlement should be borne mind.

The designer should now be in a position to decide on the type of foundation to be adopted—independent otings, raft foundation or piles. Finally, by considerg all the relevant factors in relation to the probable ethod of failure, shear or settlement, he can proceed the detailed design.

Allowable Settlement

The permissible differential settlement obviously aries considerably with different types of structure, and ere seems to be little information available for the uidance of the designer. Two principal effects of fferential settlement have to be considered: (a) the ability for cracks in ceilings and wall coverings and the arping of level floors; (b) the redistribution of reactions d moments in the members of the structure itself. he first of these criteria usually governs in ordinary uildings, while the second often applies to bridges, aducts, etc. With tanks, water towers and similar ructures conditions similar to (a) apply, the criterion eing freedom from cracks which would impair the mperviousness of the structure.

Terzaghi and Peck[18], from a study of settlement cords, state that the differential settlement of uniformly aded continuous footings and of equally loaded spread otings of approximately the same size is unlikely to xceed 50 per cent. of the maximum settlement, and that most ordinary structures such as office buildings, apartment houses or factories can withstand a differential settlement of $\frac{3}{4}$ in. They suggest that the allowable pressure should be chosen so that the maximum settlement of an individual footing is 1 inch. With flexible raft foundations, because of the smoothing out of the effects of non-uniformity of the soil, it is suggested that a maximum settlement of 2 inches is unlikely to result in excessive differential settlement.

Tschebotarioff[19] goes even further and states that as long as the total settlement does not exceed 2 or 3 inches no damage is to be expected in most buildings.

Some designers work to the rule that the maximum differential settlement between two adjacent walls or columns must not exceed a certain fraction of the span, on the same principle as the ordinary deflection limitations used in floor design. This seems a more rational approach, and it would be helpful to designers if more data were available as to the maximum differential settlement which can be tolerated with different types of interior finish. The problem of the loads and moments in the framework set up by differential settlement is considered later.

Effective Load

When designing a building for safety against shear failure, it is usual to take the maximum combination of dead and live load, as used in the design of the stanchions. In addition, eccentricity of loading due to wind pressure or other causes may have to be taken into account. For settlement analysis it would be incorrect to use the maximum combination of loads, since in most structures such a condition rarely arises, and then only for a very short time. For example, a concert hall is heavily loaded for short periods of time, with longer periods in between when only the dead load is acting. A warehouse, on the other hand, may carry its full load for a much greater proportion of the time. During recent years many observations of actual settlements have been published, compared with the values predicted from soil tests. In these, various arbitrary assumptions have been made as to the effective load. An example is given by Cooling[20] of a grain silo at Oxford. Observations of settlement were made over a period of about $4\frac{1}{2}$ years, and the intensity of loading was also recorded. The settlements showed considerable fluctuation, following closely the variations in the loading. Two theoretical settlement curves were calculated from the soil properties; one for dead load only and the other for dead load plus one-third of the maximum live load. The former slightly underestimated the settlement at the end of the period; the latter overestimated the final settlement, but covered the fluctuations more effectively.

To enable estimates of long-term settlement to be made more accurately, data should be collected of the variation of loading, over long periods of time, on different types of structure. A series of "time-average" factors could then be specified, appropriate to various classes of structure, and these could be used to determine the effective load for consolidation purposes. The effective load would thus be the dead load plus the live load multiplied by the factor. The frequency of application and the rate of build-up of the live load must also have some effect, and could be allowed for in these time-average factors.

Allowable Bearing Pressure

The *maximum safe bearing capacity* is defined as the maximum intensity of loading that the soil will carry without risk of shear failure irrespective of any settlement that may result. This is arrived at by dividing the

ultimate bearing capacity by a factor of safety or load factor, usually at least 3.

The *allowable bearing pressure* is the maximum pressure which can be applied, taking into account both shear failure and settlement. If settlement conditions govern this will be less than the maximum safe bearing capacity.

During the period of erection of a structure the effects of any differential settlement are generally compensated for in the course of construction. On cohesive soil such settlement is mainly the so-called "immediate" settlement due to elastic and plastic deformations, but, as has been previously mentioned, a small proportion of the consolidation settlement is bound to take place during the period of construction.

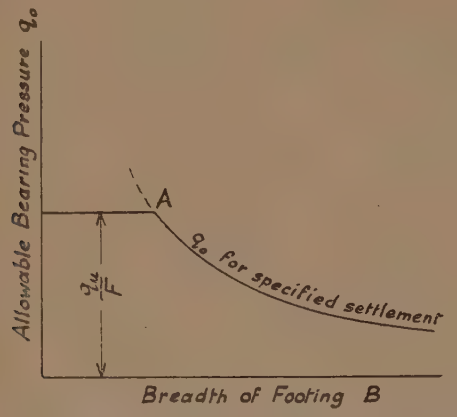


Fig. 15.—Allowable bearing pressure—cohesive soil

For cohesive soil the ultimate bearing capacity is independent of the breadth of footing B. As B increases, a greater depth of soil is affected, and the settlement increases roughly in proportion. For a given maximum settlement, therefore, the allowable bearing pressure is approximately inversely proportional to B. The variation of allowable bearing pressure with breadth of footing is shown diagrammatically in Fig. 15.

Where the foundation is on uniform compressible soil extending to a great depth a rough approximation to the greatest breadth for which shear failure governs can be obtained as follows. It has been shown that the approximate consolidation settlement is given by

$$\frac{\rho}{B} = 0.8\, m_v q$$

At the point A in Fig. 15 the allowable bearing pressure for a given settlement is just equal to the maximum safe bearing pressure for stability against shear failure. For a shallow square footing q_u is approximately equal to $6c$, and if the factor of safety for shear failure is taken as 3, the maximum safe bearing pressure is $2c$. The greatest breadth B for which shear failure governs is obtained by substituting $q = 2c$ in the equation for ρ/B.

Thus for the point A, $\dfrac{\rho}{B} = 1.6\, m_v c$

and $B = \dfrac{\rho}{1.6\, m_v c}$

If the average effective load for consolidation is taken as some fraction k of the maximum load, the above equation can be modified to

$$B = \frac{\rho}{1.6\, k\, m_v c}$$

As has been already mentioned, the settlement characteristics of cohesionless soils are difficult to determine. For a given bearing pressure the increase of settlement with increase of breadth is much less marked than with cohesive soil. The allowable bearing pressure is often taken as constant for any width greater than, say 3 feet. Thus no advantage is taken of the additional bearing capacity gained by increasing the width, but the resulting settlement is unlikely to be excessive.

Fig. 16 shows diagrammatically, for cohesionless soil, the variation of allowable bearing pressure with breadth of footing. The horizontal dotted line indicates the common practice of limiting the bearing pressure to the value appropriate to a specified breadth of footing.

The new draft Code of Practice No. 4, "Foundations," gives a table of ultimate bearing capacities for various types of rocks and soils. It is recommended that a factor of safety of two should be applied to the tabular values. The safe bearing capacity thus obtained applies to surface loading, and rules are suggested to allow for the effects of depth and breadth of footing.

It seems strange that in spite of the progress of soil mechanics the old method of specifying bearing capacities for different types of soil should persist in this Code. For the identification of the various types of cohesive soil, limits of shear strength are given. If the shear strength has to be determined in order to classify the soil, why not use if directly to find the ultimate bearing capacity? The clauses relating to the factor of safety are somewhat vague, and it is now generally accepted in this country that a minimum factor of three as regards shear failure should be specified.

In the opinion of the author, this section of the Code should be regarded in the same way as the "deemed to satisfy" clauses in building regulations, that is, as a rough guide for unimportant jobs for which thorough site investigations and soil testing are not justified.

Methods of Reducing Settlement

Various expedients can be used to keep the settlement within reasonable limits. One is the provision of deep basements. The weight of soil removed by excavation

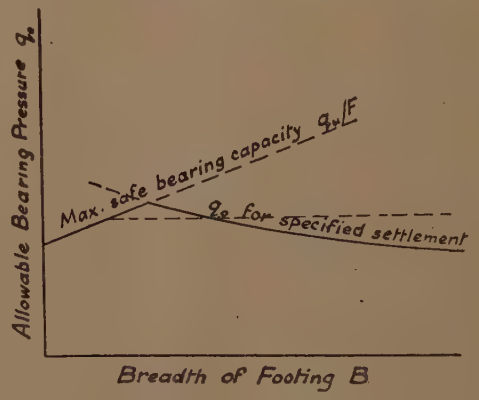


Fig. 16.—Allowable bearing pressure—cohesionless soil

reduces the net load which will be applied to the foundation when the structure is completed. A structure of which some parts are heavier than others—for example a building with a tower—should have its basements proportioned accordingly. A problem which sometimes arises, calling for special consideration, is that of a structure which is to be completed in stages, for example, where upper storeys or a tower are to be added at some future date.

Differential settlement may be reduced by designing so that heavier loads are applied round the periphery

he building than in the centre. Rigidity of con-ction may effect the same result, as mentioned later.

T Foundations

When the allowable bearing pressure of the soil is and the size of individual spread footings becomes e compared with the spacing between them, it omes more economical to design a continuous raft ndation covering the whole area. This leads to tain further problems. The depth of soil appreciably cted by the pressure, and therefore contributing to total settlement, is very much increased, although applied contact pressure is reduced by being spread r a greater area. The rigidity of the raft, as well as t of the structure as a whole, has an important effect on both distribution of pressure and settlement. om the point of view of minimising differential settle-nt a very rigid raft foundation, or a very stiff building me, would appear desirable. Such rigidity is, how-r, unattainable without using sections of uneconomic portions. In addition, on cohesive soil the stress ncentrations around the edges of the raft would almost tainly give trouble. A limited amount of differential tlement must therefore be accepted. At the other treme we have the usual assumption of uniform ward pressure, and the raft becomes in effect an verted floor. Even if the raft is flexible enough to ke up the shape of the settlement profile for uniform ading, the flexibility of the individual beams and bs of which the raft is composed will bring about al variations in contact pressure. The bending oments and shears in these members will thus differ om the values predicted on the assumption of uniform ading. Another uncertain factor is, of course, local riation in the soil properties.

The designer is faced with the question of what essure distribution he should use in calculating his nding moments. The Code of Practice is not very lpful on this point. It states (p. 66), " The distribu-on of pressure by a raft over a large ground-bearing eds careful consideration. In some cases ground essure will tend to be greater toward the centre of the ab and in others less. The distribution of bending oments and shears in the raft are therefore not capable precise determination. Reinforcement should be ovided to resist all the tensions likely to be induced." he last two sentences appear a little inconsistent.

Various cases of beams and slabs resting on elastic edia have been investigated mathematically, and the sults are to be found in treatises on Theory of Elas-city. These are of little use for foundation problems cause of the non-elastic properties of soil and the certainty of all the data. Attempts have been made, wever, to use the pressure/settlement relation of the il to calculate the bending moments on the foundation ructure. One of these is the " soil-line " method of ofessor A. L. L. Baker[21]. In this the supporting rce at any point is assumed to be made up of three mponents : that due to uniform bearing pressure at due to the deflection of the foundation beam or ab, and that due to the compression of the soil. One both of the last two may of course be negative ; tegrated over the whole area they cancel out.

For the purpose of airfield pavement design the essure/settlement relation is usually expressed in the rm of a *modulus of subgrade reaction*[22], which is defined the resistance per unit displacement offered to a aded bearing plate of specified size, measured over a nge of 0 to 0.05 in. vertical displacement. This range quite arbitrary and bears no definite relation to the range of loading required to produce a shear failure. The modulus is therefore of the nature of an empirical coefficient rather than a true elastic modulus.

A special case of raft foundation is that of an oil tank. Here the downward load must be uniform, however irregular the distribution of the supporting pressure may be. The structure is frequently called upon, unlike most structures, to carry its maximum design load. Also, the development of slight cracks, which in other types of building would be of little consequence, cannot be tolerated. To add to the difficulties inherent in oil tank design, these structures usually have to be constructed on sites adjoining river estuaries where the soil conditions are particularly unfavourable. Terzaghi, in the James Forrest Lecture, 1938[23], cites an instance of an oil tank 100 feet in diameter which he designed, founded upon a bed of soft clay. The tank, constructed of welded steel plate, rested on a lightly reinforced concrete slab 4 inches thick, which deflected to the natural bowl-shaped settlement induced by the uniform load. A similar tank, by another designer, had a heavily reinforced slab stiffened by deep ribs. This raft was too stiff to adapt itself to the differential settlement, and fractured along two diameters.

A form of construction which has been used successfully for the base of a tank is that of a flat inverted dome with heavy peripheral reinforcement. This has proved an economical method of combining strength and stability.

Eccentric Loading

The design of eccentrically loaded foundations involves uncertainties such as the distribution of contact pressure on the soil and the criterion of failure. The usual method of procedure is to assume linear variation of pressure and to design the footing so that the maximum allowable bearing pressure is not exceeded. The probable pressure distributions on clay and sandy soils are of the types shown in Fig. 1 (c) and 1 (d) respectively, but data for exact computation is not available. It would appear from these diagrams that in the conventional methods of design the bending moments in the slab tend to be underestimated if the soil is clay. The converse is the case with sandy soils.

Wherever practicable eccentricity of loading should be avoided, but there are many cases where this is impossible, as for example the foundations of retaining walls and bridge abutments. Hamilton[24] has pointed out that since complete failure of an eccentrically loaded foundation involves tilting of the structure the passive resistance of the soil bearing against the side of the foundation is brought into play. He makes the suggestion, based on Fordham's experiments at Swansea, that half the maximum passive resistance may reasonably be taken into account as contributing to the stability of the footing. This assumption has the effect of shifting the position of the resultant force on the base, so that the eccentricity is reduced.

The problem of eccentrically loaded foundations is being investigated at the Building Research Station. Some analytical work supported by experiments using photo-elasticity has been carried out at Newcastle, but the results have not yet been published.

Miscellaneous

Many special problems arise in foundation design in which knowledge of the principles of soil mechanics is of great value. Space prohibits more than a very brief mention of a few of these.

The seasonal shrinkage and swelling of clay soils due to changes in moisture content are a frequent source of

trouble with shallow foundations. The transpiration of moisture from the soil by vegetation is an important factor in producing these changes, and the drying effect of large trees is well-known. The obvious remedy is to carry the foundations down to a depth sufficient for the seasonal variation of mositure content to be negligible. On certain clay soils in this country this depth may be as much as five feet. Alternatives include the use of short bearing piles, a method developed at the Building Research Station[25], and the construction of impervious aprons several feet wide round the perimeter of the building to limit the changes in the moisture content of the soil under the footings of the walls.

Some interesting data on this subject and on other causes of settlement of structures are given by Cooling and Ward[26] in a paper presented at the Rotterdam Conference.

Influence of Soil Mechanics on the Design of Structures

The effects of the rigidity of the structure on the distribution of the load to the soil has already been discussed. Unless the structure is flexible enough to take up the shape of the settlement profile, redistribution of reactions and moments is inevitable.

Meyerhof[27] has applied mathematical analysis to the determination of the effects of differential settlement on rigid and semi-rigid building frames. The general idea is as follows. The primary forces and moments in the frame are determined in the usual way, neglecting settlement. Settlement moments and forces are then calculated in terms of differential settlement. By relating the deformation characteristics of the frame to the settlement/pressure curve for the soil the numerical values of the settlement moments and forces are computed. The final stresses are obtained by adding algebraically the initial and settlement stresses. For any particular member, the initial or final value, whichever is the greater, is used in the design. It is suggested that for all buildings except those used for storage the effective load for settlement should be taken as the dead load plus half the maximum live load.

The method of indirect model analysis has also been applied to this problem. It may be argued that the time and labour required for such mathematical analysis or for experiments with models are not justinable in view of the uncertainty of the data. It is true that the effective loading is a matter for conjecture, while the properties and uniformity of the soil can only be approximately assessed. Such analysis can, however, indicate limits outside which the stresses are unlikely to vary, and careful study of a large number of examples, correlated with experimental data, may eventually lead to simplified approximate methods.

The advocates of the plastic method of design claim that in a rigid steel frame considerable differential settlement can be tolerated without reducing the load factor, because of the formation of plastic hinges. An example is described by Horne[28] of tests on continuous beams in which differential settlement of the supports had little effect upon the collapse load. It is questionable, however, whether the possible formation of plastic hinges is desirable in buildings, in view of cracks in walls and ceilings and other undesirable consequences. Clearly this is a point on which further information is required.

For buildings covering a large area of compressible soil the ideal is probably a happy medium between a rigid structure which experiences considerable secondary stresses and a pin-jointed frame which may prove overflexible. One method for achieving this is to construct the building as a series of fairly small rigid units, flexibly connected to one another.

It is clear that knowledge of the soil conditions must have considerable influence, not only on the type of foundations necessary, but also on the general design of the structure itself.

Future Developments

Soil mechanics has done much to help the structural engineer to avoid failures due to foundation troubles, and to enable him to design more economically. A considerable amount of knowledge has been accumulated about the shear strength, compressibility and bearing capacity of soils, and fundamental research on these properties is still in progress at the Building Research Station, at Imperial College, London, and elsewhere.

Much of the experimental work on bearing capacity and settlement has been carried out in the laboratory on small-scale models. In interpreting these results, as in all model tests, due attention must be paid to the "scale effect." A point sometimes overlooked is that in a laboratory test the soil may be more uniform than in the relatively large mass of soil beneath a structural foundation.

The loading or plate-bearing test, the oldest of all soil mechanics tests, comes in for a great deal of criticism. It must be borne in mind that this, too, is a form of model test, though on a larger scale than laboratory tests and applied to the soil in its natural state. If the test is not carried out with considerable care, or if the results are not interpreted in the light of soil mechanics principles, the results can be very misleading. Nevertheless, this form of test has an important place in foundation engineering. It is essential that a carefully standardised procedure should be followed, since irregularities in loading and unloading or insufficient time for plastic deformation to develop, may invalidate the results. A recommended procedure is given in the Code of Practice[5], "Site Investigations," Appendix J, and some of the specialists in site investigation have their own standard methods of performing the test.

As has been emphasised on many occasions by the leading authorities on soil mechanics, the greatest need of to-day is the correlation of theory and laboratory test results with observations on actual structures, both during and after erection. Settlement records are not difficult to obtain, and a considerable amount of data has already been accumulated. On the other hand, examples of total collapse are, fortunately, rare, and when they do occur it is naturally rather difficult to analyse the conditions which have caused the failure.

This brief review of the present position of soil mechanics in relation to certain aspects of structural engineering reveals a number of gaps in our knowledge. The following is a summary of specific points, on which more information would be of great value to structural engineers.

(a) Tables of average soil properties (shear strength, compressibility, etc.), to be expected from soils as classified according to the Code of Practice "Site Investigations," Table I, under specified conditions of water content, etc.

(b) More data regarding the compressibility of sandy soils, and suitable field and/or laboratory tests for determining these properties.

(c) Data regarding the "time-average" factors to be used for estimating effective loads for settlement analysis.

(d) More definite data regarding the contact pressure distribution and conditions of failure for eccentrically loaded footings.

(e) Simplified approximate methods of estimating condary forces and moments in rigid and semi-rigid mes subject to differential settlement.

(f) More experimental data of the effects of differential ttlement on steel frame structures in relation to the astic theory of design.

(g) Data for the economic design of spread footings d raft foundations, taking into account the flexibility the footing or ratt and the variation of contact pressure th deflection of soil surface.

(h) The development, for certain classes of building, materials and methods of construction for walls, ors and ceilings which will withstand much greater fferential settlement than can be tolerated under rmal conditions.

It is only by close co-operation between practising gineers, soil specialists and research workers that these roblems can be investigated, and that knowledge of il mechanics can be more widely disseminated and tilised in the most efficient way.

In conclusion, the author wishes to express his thanks the various friends with whom he has discussed the bject-matter of this paper and from whom he has ained much valuable information and helpful criticism.

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Hangars at London Airport—Design of Large Span Prestressed Concrete Beams

Discussion on Mr. A. J. Harris' Paper*

Mr. H. KAYLOR (Associate-Member) asked whether here was any special reason for deciding that the test oad should be equal to the design load. He would ave thought that 1½ times, or at any rate some bigger oad than the design load, would have been applied as a est.

Secondly, he invited Mr. Harris to describe the method sed for the assembly of the elements forming the econdary beams, when grouting up between the ements. What precautions were taken to prevent the ortar seeping into the holes and grouting up the ables?

Mr. HARRIS, replying to the question concerning the est load, said that the test load amounted as it was to 88'; furthermore, there was a danger that a 50 per ent. overload would overstrain a structure of this cale somewhere or other. The extra expense would have been considerable, and in fact the clients were satisfied with a test of the full working load.

To keep the jointing mortar off the cables, the cable ducts were bell-mouthed and a short length of tube slipped into the bell-mouths; the cable passed through this tube.

Mr. DONOVAN H. LEE (Member of Council), said Mr. Harris had presented, as usual, an excellent paper. But he asked if Mr. Harris would think over the answer he had given to Mr. Kaylor's question, because he felt, as the question had inferred, that 1½ times the design load was the sort of test load one expected to have imposed on a prestressed beam. Structural engineers were asked to design for a certain applied load, and in some cases repetitions of 1½ times that load could occur. Mr. Lee preferred the first answer, that an overload test would have cost a lot more money, rather than an opinion say, that such a test was to be avoided in this case.

Mr. HARRIS said he had mentioned the figure of 6,000 lb./sq. in. for the concrete.

So far as the imposition of 1½ times the working load was concerned, it depended on how much confidence

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one had in the British Standards Institution and other bodies responsible for drawing up codes of practice. Only if one did not believe the loads they specified, would it be wise to over-design. But in a structure of this nature, in order to impose the working load there would have to be the regulation thickness of snow on the roof, a gale blowing into just the right parts of the structure and, coincident with that, a full crane load moving in the right direction. That was a possible combination of circumstances, but not, he felt, a very likely one ; a 50 per cent. overload seemed virtually impossible. In a very large structure of this sort the design live-load was in any case relatively unimportant ; a prestressed concrete structure has a very high fatigue limit and it would carry the working load indennitely—at least nobody had yet succeeded in impairing a prestressed beam as the result of repetitions of the design load, and many had tried.

Dr. K. HAJNAL-KONYI (Member), first congratulated Mr. Harris on his paper, which contained much useful information.

Having intended in the first place to deal with the question raised by Mr. Kaylor, Dr. Hajnal-Konyi pursued further the line taken by Mr. Donovan Lee. He pointed to the table of stresses on page 232, according to which, after all losses had been accounted for, there still remained a minimum compressive stress of 127 lb./sq. in. at the bottom. That was the worst possible case, and it could happen only after all losses of prestress had taken place. He did not know how long after the prestressing the load was applied, but he had to assume that there was still a considerable margin as against the maximum loss of prestress. The author has assumed a total loss of 15 per cent., i.e., about 600 lb./sq. in. It was not an unreasonable assumption that, when the test loading was carried out, the stress under the full design load at the bottom would have been not less than 200 lb./sq. in. The live load produced a tensile stress of 768 lb./sq. in. and half of that was 384 lb./sq. in. ; therefore, taking off 200 lb./sq. in., it would produce a tensile stress of less than 200 lb./sq. in., which was so low for a concrete of 6,000 lb./sq. in. crushing strength that, if the author's design assumptions were realised, the structure would not have cracked under 1½ times the design load. Under 1¼ times the design load no tensile stresses should have occurred at all.

Dr. Hajnal-Konyi was of the same opinion as the previous speakers, that the test did not prove anything. What we wanted to find out from a test was whether the structure was carrying the load according to the assumptions of the design ; if it did, even a substantial overloading could not have caused cracking.

Studying the figures of deflection, he had come across a very peculiar feature of the test results. At the end of the paper the author had given four figures, recorded on four consecutive days. The difference in time between the first and second readings was 19 hours, the difference in deflection was .0165 in., which gave a rate of increase of .00097 in. per hour. The difference in time between the second and third recordings was 24 hours, the difference in deflection was .0295 in., the rate of increase being .00123 in. per hour. The difference in time between the third and fourth recordings was 24 hours, the difference in deflection was .0695 in., and the rate of increase was .00290 in. per hour, i.e., more than three times the initial rate of increase.

Having carried out many tests, he was rather puzzled by these results, because the increase was due mainly to creep and the nature of creep was that the rate slowed down with time, whereas in this case it speeded up, or seemed to do so. He asked why Mr. Harris did not pursue the matter further, leaving the load on for a week or a month or even longer, in order to see what happened. This would have hardly increased the cost of the test.

Coming to the question of the factor of safety, he said that the table on page 232 gave certain stresses under different conditions of loading, but did not reveal anything about the factor of safety. Whilst the working stresses in ordinary reinforced concrete were an indication of the factor of safety, the stresses given in the paper bore no relation whatever to the factor of safety, so that we did not know on what assumptions the design was made. There was a First Report on Prestressed Concrete issued by the Institution of Structural Engineers which recommended a factor of safety of 2 on the overall design load (i.e., the dead plus live load). Although Dr. Hajnal-Konyi was a member of the Committee responsible for that recommendation, he was of the opinion that it might have gone too far in this respect and that it would have been probably better to distinguish between dead load and live load, allowing a lower factor on the dead load, which could not be increased, and perhaps a higher factor on the live load. In the structure described in the paper the overall factor of safety was less than 2. This was not a criticism and he was raising no objection since the different load factors for dead and live load respectively resulted in an overall factor smaller than 2 when the ratio live / dead load was high, but he would be interested if Mr. Harris would be good enough to reveal what factor of safety he had assumed and how he had arrived at it.

Mr. HARRIS, dealing with the first question raised by Dr. Hajnal-Konyi, said it seemed to him to be unnecessary to ask under what load the beam would have cracked. As to the suggestion that the test made on the structure proved nothing, he said it had proved precisely what it was intended to prove, that the structure would carry without signs of distress the load for which it was designed.

With regard to the deflection figures, they were taken over a period of three days, during which the weather varied considerably. He also had detected the curious increase in the figures ; they meant nothing whatsoever without measuring at the same time the moisture in the air, and the temperature. The purpose of the test was not, to his great regret, to carry out an interesting test on the beam ; nobody would have been better pleased than himself to have made such a test, but unfortunately the authorities did not share that view. They wanted to use the structure.

He had indeed checked on the factor of safety, and the figures are given in the written reply. The structure was of a type in which the factor of safety was best assured, not by such-and-such a load factor, but by a safe stress design. That was the design asked for and provided. Nobody assumed that the structure would be required to carry 1½ times or twice its working load ; nobody could get that load on—there was difficulty in applying even the working load. He repeated his view that a severe overload carried out for test purposes would be unwise from the user's point of view.

Mr. JOHN FABER (Associate-Member) added his appreciation of the paper. He thought that the paper might well be regarded as a model paper, and indeed a masterpiece of concise expression.

The conception of the hangars described was brilliant. The idea of casting the main beams *in situ* around the precast diaphragms was perfect, and he considered that the arrangement of the secondary beams, precast in

ctions and then post-tensioned, was also perfect. It as easy to see something after it had been done and y that it was the logical answer : but starting from ratch and having to develop such a project in one's ind was quite a different matter.

With regard to the stresses in the concrete, the paper d not describe the concrete mix, but it was stated iat the working stress was taken at 2,000 lb./sq. in., ased on a required crushing strength of 6,000 lb./sq. in. was very difficult to relate cube strength to the rength of concrete in actual structure, and Mr. Faber id he believed there was very little knowledge on that atter. The cubes were prepared usually under very ose supervision by the foreman, and were cured most refully ; but the foreman was not always around when ie concrete was placed in every part of every beam, and ut on the job there were various difficulties in the way curing, as for example frost, rain, sunshine, drying inds, and so on. So that the concrete cured under far orse conditions when in the structure than when in the rm of cubes.

Furthermore, cube strengths were often misleadingly igh, due to the strong shape of cubes. Even cylinders iled at about 0.7 times the stress at which cubes failed. nd Mr. Faber believed that the strength of the concrete the 4 in. walls of the box girders would be considerably ss than in a 6 in. cube.

We also had to bear in mind the variability of concrete. collection had been made recently of cube strengths om thousands of contracts carried out in this country the past five years. Confining ourselves to the ordiary 4 : 2 : 1 mix, although the Code required a strength only 3,000 lb./sq. in. for such concrete, many of the st results were as high as 6,000 lb./sq. in., and some ven as high as 9,000 lb./sq. in. But what was very uch more worrying was that some results were as low 2,000 lb./sq. in., and these tests were made on conetes used on high-class jobs, supervised by trained esident Engineers. Mr. Faber asked if Mr. Harris ould enlarge on the question of the stresses in the oncrete and how one could be really sure that the oncrete in the structure he had described had a strength f 6,000 lb./sq. in.

On the question of wind allowance, he understood om the paper that it was considered desirable, even at ese high stresses, to allow a 33⅓ per cent. increase in llowable stresses caused by wind. This meant allowing stress of 2,700 lb./sq. in., which was 45 per cent. of the ,000 lb./sq. in. shown by the cube tests but which he ought might not be available in the structure itself.

Dealing further with the wind forces allowed for in esigning the structure described in the paper, Mr. Faber aid he had read out of the paper that when the wind lew horizontally it must of necessity also lift the roof. t seemed that that could be inferred from Chapter V, ut he was not sure whether it should be inferred. He onsidered that the upward force on a roof should be uarded against, but he did not think we should rely on —winds could be too gusty and temperamental for hat. According to his own figures, if we considered a orizontal wind blowing on the main girder over the oor, and ignored the uplift, a tensile stress of about 25 lb./sq. in. might result, and he asked whether Ir. Harris would consider such a stress allowable.

Coming back to the 4 in. walls of the box girder, he aid it was stated by Mr. Harris that the diaphragms in he box girders, being at 15 ft. centres, were considered ufficient to ensure that the 4 in. walls were safe against uckling. Even with those diaphragms, the 4 in. walls ere standing in free panels 14 ft. by 14 ft., with quite onsiderable stresses in the concrete. Buckling was a difficult problem ; it was liable to occur if there were an initial curvature in any of the members, or, when using a material like concrete, if the concrete across the members was not entirely homogeneous. He would be grateful, therefore, if Mr. Harris would explain how it was decided that 14 ft. by 14 ft. panels were satisfactory.

Mr. Faber added that Mr. Harris's excellent paper had greatly increased his very warm regard for the author and his work.

Mr. HARRIS was most grateful to Mr. Faber for his appreciative remarks.

With regard to the concrete cubes, he said the standard methods of making and testing the cubes were followed. He had not mentioned the results in the paper because it was concerned with design rather than with construction. From his limited knowledge—and he did not wish to extend himself on the matter—the cubes were good ; and certainly the care which the contractors exercised in placing the concrete was exemplary.

It was true that the strength of a cube gave little indication of this strength of the concrete in the structure. It worked both ways ; when little care was taken over compaction, the cubes were better, but on the other hand when vibration was associated with pressure and with steam curing it was extremely difficult to give the concrete in the cube the same treatment. He knew of one job where the cubes at 28 days gave 8000-9000 p.s.i. regularly, but where cylinders cut out of the finished elements at 90 days gave 11,000 p.s.i. Where small precast elements are employed, it is occasionally possible to test a complete element, but this is not often practicable and we must wait, it would seem, for some non-destructive method of testing, such as the ultra-sonic sounding method designed by the Road Research Station, to be developed and accepted.

With regard to the hozizontal wind stress, he said the Code of Practice permitted the increase of stresses employed, and he would not attempt either to criticise or to justify the work of the eminent gentlemen who had drawn up the Code. On the question of the uplift and the various stresses arising under the extreme conditions of which Mr. Faber had spoken, Mr. Harris said that the full range of possible alternative directions of wind load according to whether doors were open or shut, etc., had been considered and he had to admit that he had not been able to detect the tensile stresses of which Mr. Faber spoke.

Some calculations were made concerning the buckling of the webs, but he would hesitate to estimate their value ; it was indeed an estremely complicated subject. It had arisen in terms of prestressed concrete beams in the case of the hangars at Melsbroeck, and in the proceedings of the prestressed concrete congress at Ghent there was a theoretical treatment of the buckling in the webs of these beams. The situation there was somewhat different because the prestressing wires in that particular case were inside the hollow beam and were in no way in contact with the concrete, except at the diaphragms. In the case described in the paper the concrete was stabilised by the cables embedded in them.

Mr. Harris summarised by saying that, if he were asked why it was thought that 4 in. was good enough, he would give the engineer's standard answer to the effect that the thing looked right and it could not be proved that it was wrong.

Mr. O. J. MASTERMAN (Associate-Member), commented that Mr. Harris, in his usual light and agreeable manner, had tossed off the work described in the paper almost as a trifling achievement. But having seen the job, and a lot of other prestressed jobs in this country and abroad,

Mr. Masterman said that in his view it was a very great achievement ; it was a very impressive example of prestressed work.

It had been said very rightly by Mr. Harris that the effect of the secondary beams had been spoiled by the lighting put underneath them. Nevertheless, Mr. Masterman recommended that all who were interested in such work should see the structure, because the 110 ft. beams with a 4 in. web and a 4 in. flange were really something worth looking at ; the design was as courageous perhaps as any other prestressed design which had yet been erected.

Drawing attention particularly to the slim sections in the main beams over the hangar doorways and in the secondary beams in the roof, he said they illustrated again the second notable characteristic of prestressed concrete, the first being flexibility. We were already getting to the stage at which the sections were comparable with steel sections ; it would be seen that in the job described, if we added up the thicknesses in relation to the depths, they compared with steel beams. It was the same in the case of the Karachi hangar building, where he believed a 22 ft. depth of beam had a 9 in. web.

Drawing attention again to the point raised by Mr. Faber as to where we could stop in designing thin sections, he recalled a remark by Mr. Harris that the 4 in. web was about thin enough, and that he had made some buckling calculations. Incidentally, in the main building there were not any cables in the depth of the web at the centre to prevent buckling, and he asked for some guidance as to how we should know just how far we could go in making sections thinner and thinner.

Another question was why the author used a modular ratio of 8. The author might have guessed that the concrete would show at about 5-6 million, and it did in fact come out at about 5 million in the deflection tests at the end, and one would have thought that a modular ratio of about 5 would have been more correct.

Finally Mr. Masterman said that, as would be expected from Mr. Harris, the paper was wonderfully clear and was vey nicely presented in print.

Mr. HARRIS, dealing with the question as to how far we could go in slimming sections, said that in the case of a web with a cable passing through it, he considered that the limit was 4 in. There was a certain size of cable and it was necessary to have concrete on either side, not just a thin layer of cement, and even a little mild steel, and he did not think that less than 4 in. was practicable for *in situ* work, though for precast work in special circumstances we might get down to a thickness of 3½ or even 3 in.

On the question of the modular ratio, the modulus of the concrete under a long-term load would almost certainly fall to about 4×10^6 or even less, and he thought the ratio used was about right.

Returning to the webs at mid-span, it should be remembered that the cables have absolutely no effect in stopping buckling under external forces ; the value of making them solid with the concrete is that the cable force itself cannot then cause buckling.

Lieut.-Colonel R. F. GALBRAITH, R.E., M.C. (Vice-President), added his meed of praise of a most interesting and excellent paper, and supported Mr. Harris with regard to his safe design stress theory, for he believed that the safe design stress was the critical thing in which we were interested. If we wanted 1½ times the working load on the beam, we should design it for 1½ times. He did not think that for practical buildings, which were going to be used, we were really interested in ultimate conditions ; those conditions were of importance in research work, but not for practical design.

Dr. P. W. ABELES (Member) also very much appreciated the paper. He agreed with Mr. Masterman that the secondary beams were some of the finest examples of prestressed concrete construction ; he had had an opportunity of seeing them from underneath, before the bottom was thickened, and they looked very elegant with their narrow web.

Referring to a statement by Mr. Harris that the " T " beam was not economical, Dr. Abeles said that he did not agree. He considered the " T " beam to be one of the most economical forms, which could however be used only if the construction depth was not limited. In a "T" beam, firstly, the centroid was very high and we were able with a relatively small prestressing force to produce a large pre-compression at the bottom fibre. Secondly, because the depth of the " T " beam was much greater than that of an " I " beam, a smaller reinforcement was required for ultimate load, always provided that the cables are placed near the bottom fibre. When the original loading for the secondary beams was reduced, it would have been possible in Mr. Harris' case to keep the " I " shape and reduce the depth, but this would have most likely been a less economical solution, otherwise it would have been employed.

After having given full consideration to the safe design conditions, he agreed that with a roof like this a smaller factor of safety might be permissible than the factor two required in the " First Report " ; but a certain minimum factor of safety of, say, 1.5 was certainly required and this seems to be obtained in Mr. Harris' design. However, Dr. Abeles was, on principle, unable to agree with Mr. Harris that contrary to the requirements of the "First Report" ultimate load conditions could be disregarded and that a safe working load stress design was sufficient for the design. We might have, for example, three different sections, each designed for the same permissible working load concrete stress but for different loading conditions. One of these might have a factor of safety against failure of 4, the second of 2 and the third of 1.2. There was no relationship whatsoever between the concrete working load stress calculated for a straight line stress distribution and the stress under failure conditions, as with other materials with which a permissible working load stress already ensured a minimum factor of safety against failure. Consequently, with a material as prestressed concrete there must be separate designs for working load and for failure conditions.

Dr. Abeles agreed with Mr. Harris that with difference of temperature and sunshine the magnitude of deflection was affected and that in this special case a much higher load was not so essential, but he agreed also with the other speakers that " on principle " generally a higher test load ought to be applied than the working load to see whether the efficiency of prestress is ensured. There was always a possibility that the prestress force was less than specified owing to friction losses exceeding the assumptions or other reasons. The tests would prove that the execution agreed with the assumptions if the loading corresponded to a concrete tensile stress of, say, 75 to 80 per cent. of the modulus of rupture, when cracking ought to be avoided.

Finally, Dr. Abeles mentioned that the notation used by Mr. Harris was not quite in agreement with that of the "First Report" prepared by the Institution's Committee of which Mr. Harris was a member. In the Report the lower fibre was called 1 and the upper fibre 2 ; Mr. Harris had reversed them. The distance of the outer fibres

m the centroid were called " e," while Mr. Harris used e symbol " v," and with regard to the section modulus, tle " z " signs were used instead of the capital " Z." nsequently, instead of v_1, v_2 z_1, and z_2 the respective mbols e_2, e_1, Z_2 and Z_1 apply.

Mr. HARRIS said one reason why he had stated that T " beams were not found to be economical was that ich had been given by Dr. Abeles, that they were ways very deep. That was not always a serious jection, because nine times out of ten in a civil en- neering structure, particularly one of the type de- ribed in the paper, the only thing we could get free as headroom. There was the further point that the T " beam under load gave a very high stress variation the bottom fibre, and that limited the design rather sely.

The problem of the factor of safety was rather too ge for discussion at length at such a meeting. How- er, it did depend on the nature and scale of the struc- re. If it were a beam spanning 15 ft. and carrying ane traffic, there was a very great possibility that an tremely heavy overload would occur. In such a case e would provide a high factor of safety, or would ovide means to relieve overload to a neighbouring am. But in the case of a large structure, where live ads were small, the question was very different ; r. Abeles had said that one could design a structure by e safe stress method which would fail at 1.1, and deed, it could be done, though with difficulty. Equally suggested that one could design a structure with a ctor of safety of 3, and it would not last for nine onths, because of corrosion of the steel in cracked ncrete. There was obviously a place for design by ad factor. There was also a very wide range of uctures where one was not interested in it ; for stance, who knew how much weight would be neces- ry to bring down the Forth Bridge ? And having rked it out, could we get that amount of weight on it ?

Mr. T. BEDFORD (Member), diverting from the chnicalities of the paper, asked if the author uld say anything about the cost of the structure scribed, for he always felt that, however interesting structure might be as an engineering achieve- ent, it was important to know how much it would st. Only very vague reference had been made to e fact that schemes for structures in steelwork were tained, and we were led to assume or to believe that e structure described in the paper was the right solution d the most economical one. As Mr. Harris had said, hangar was a building the main requirement of which as to cover a very large area, to keep out the weather. et, looking at the concrete building, there were tre- endous masses of concrete held high in the air, largely r that purpose. As a rough guess, it would appear at the weight of the complete concrete structure would somewhere about 75-100 lb./sq. ft. of covered floor ace, whereas in a steel structure one would imagine the uipment weight to be not more than 20 lb./sq. ft. It uld appear that the front girder, of 150 ft. span, eighed at least 150 tons.

There was that great mass of concrete, very in- niously designed—and he did not wish in any way to tract from the very pleasing and very nice application prestressed concrete to hangar construction—but he ll felt sceptical about the suggestion that the structure as more economical than it would be in a much lighter rm of construction.

Mr. HARRIS replied that he could not give information the cost of this structure, because he did not know it, but he was assured that the economic advantage in capital cost was real and in its favour, quite apart from the saving in maintenance costs and quite apart from the fact that steel, a material required so urgently for uses where it cannot be replaced, was economised in very large quantities.

The weight of the total load-bearing structure was, as mentioned in the paper, the equivalent of about 3.75 inches average thickness of concrete, i.e., about 48 lb. per sq. ft. or approximately 2.5 times the figure Mr. Bedford quotes for steel. Now, bearing in mind the basic costs of steel and of concrete, a rough calculation will show that the proposition that prestressing was cheaper is far from being ridiculous.

Mr. T. O. LAZARIDES, who said that he found it difficult to express his praise of the paper in terms which had not already been used, recalled Dr. Abeles' disagreement with a statement by Mr. Harris that the " T " beam was not always the most economical form. Having had occasion to carry out a very complete investigation of the economics of the thing, his conclusion was that the most economical shape of beam depended essentially on the ratio between live and dead load, and therefore it depended essentially on the span also. Even where headroom was free, as it was sometimes, the " T " beam was still not necessarily the most economical shape ; and where headroom was limited, the " T " beam was very seldom the most economical.

Mr. L. SCOTT WHITE, O.B.E. (Past-President), said he was most appreciative of the organisation of the site work which had enabled a job of such magnitude and complexity to have been executed in so short a time. In view of the large number of precast members used in the work, he would like to know what was the rate of damage to precast units, both normal and pre-tensioned. He would also like to know what troubles were experienced in post-tensioning, whether caused by over-stressing, defective wires, defective anchorages, etc. In a comparatively new but rapidly progressing system of construction of this type he considered information on these points would be most valuable.

Mr. P. B. STEER (Associate-Member), commenting that engineers had to meet the requirements of London County Council Surveyors, District Surveyors, and so on, referred to the jointing of the units of the secondary members. He asked if Mr. Harris would be satisfied to carry on that operation on any ordinary job without any super- vision or test of the quality of the mortar. He believed Professor Magnel had made tests of the mortar in joints, but it seemed that there were no tests in the course of the work described in the paper. Would it be possible to do that on any ordinary job, where perhaps the supervision was not so good as in that particular case ?

Mr. HARRIS said the technique described for jointing the precast elements had in fact been used quite widely in this country and elsewhere. The joint was not invariably of the type used in that particular job ; in some cases there was a cast joint, in other cases a grout injected joint, and in other cases no jointing material at all was used. Care was needed, but in the ordinary job the biggest stresses occurred at the time of pre- stress, which punishes carelessness and gives a certain degree of comfort to the man who has to use the structure eventually.

It would be idle to pretend that, on the many jobs with which he had been in contact, there had been no trouble with joints, but it was found that, with reason- able care and attention the method gave full success.

Mr. TIETZ (Student) asked whether calculations were made for ultimate failure, what assumptions were made for bond, and also, on the secondary beams, what assumptions were made for the efficiency of the joints. What factor of safety was arrived at on the basis of these calculations ?

Mr. HARRIS said the assumption made for bond in calculating the primary beams was, that there was complete bond between steel and concrete. That was an assumption which was borne out in practice by quite a number of tests ; various designs of piles might be mentioned—after they had been driven, the tops of the piles had been cut off and it had been found that the wires did not slip, some piles had been driven which relied solely on the bond provided by the injected grout. A large number of beams had also been broken, showing no slip of the wire. Necessarily, however, the number of beams of the size of those used in the hangar at Heath Row that had been broken was small, and any rupture load calculation must be hypothetical. The results of such calculation are found in the written reply.

Dr. E. H. BATEMAN (Member of Council), asked whether anybody had ever calculated the ultimate strength of a structure of that type.

Mr. HARRIS commented that that was a very relevant question. He took leave to doubt it. He further took leave to doubt whether a full design load test was often applied to such structures.

Mr. R. G. GRAHAME said it seemed to him that designing on a safe stress in the case of prestressed concrete, particularly in the case of a tee-beam, was not an entirely satisfactory method where there was liable to be overloading, because it appeared that the main danger was not the overstressing of the concrete in compression, but in tension, where a comparatively small increase of load could overcome the small compressive stress on the tension side of the beam.

He also asked what period of time had elapsed after the main beams were prestressed and before the loading test was carried out, because it seemed to him that the initial creep in the concrete might have something to do with the deflection, and the accelerating deflection.

Mr. HARRIS replied that the essence of prestressed concrete design is to apply a compressive stress sufficient to lower the tensile stresses under load to a safe figure. The bigger the tensile stresses, the bigger the initial compression and this is the factor which renders T-beams uneconomic except when dead load, which decompresses the lower flange, is large. Designing on a safe stress basis the factor of safety against cracking is independent of the shape of the beam but depends solely on the design stresses assumed and the tensile strength of the concrete.

To the best of his recollection, the beams were tested about a month after being stressed. The reason for the variation of deflection was to be sought in a combination of plastic strains under load and the variation of atmospheric and temperature conditions.

Mr. D. H. NEW (Associate-Member), who was invited by the President to speak, said how much he had enjoyed hearing his friend, Mr. Harris, being shot at from all directions, and noting how well he had staved off the attacks.

Speaking of test loading, Mr. New suggested that an increased test load might possibly have opened up the joints, at the vertical planes between the concrete and the rammed packing. He did not think that this would be desirable, and since no request was received for an increased loading, presumably the client did not think so either.

In the type of work described in the paper it was very necessary to have high-class workmanship. As Mr. Harris had said, a failure on one of the secondary beams had occurred soon after the first test loading on the beams had been successfully applied. After careful consideration, and the application of the most modern methods of investigation, it was not possible to give a proved reason for the failure. The work proceeded without modification in design, it being assumed that a combination of circumstances unlikely to recur was responsible for the incident.

It had been most interesting to hear the various theories on safe load design, ultimate strength, and so forth, and after that discussion one would be interested to see whether or not the structure stayed up ! His opinion, and presumably that of Mr. Harris, was that it would.

Some secondary beam units were rejected, and some repaired on site, and although he could not give figures, he could say that the numbers were relatively small. There were also some of the usual troubles associated with the use of prestressing cables, but considering the magnitude of the work, these were also small.

The President expressed gratitude to Mr. Harris on behalf of the Institution, and the meeting then closed.

Written Discussion

Professor R. G. ROBERTSON (Member) writes : Inspiration for the design of the beams for this hangar could be found in the aircraft which it housed, and the writer would have liked more details of the calculations ; the tabulated beam stresses appeared to be based on the final cable pull after relaxation, for which 15 per cent. was allowed but the stresses in the concrete due to this increased initial cable pull were not given, and the stresses in the secondary beams were already higher than specified.

If 15 per cent. were added to the cable pull for subsequent loss it might be anticipated that an appreciable portion of this added pull would remain even after erection, and the tabulated stresses under the head " PS " would be increased by a similar percentage.

When allowance was made for a 15 per cent. increase in cable pull the lower fibre stress in the secondary beams would seem to be 2850 lb./sq. in. instead of the 2000 lb./sq. in. specified ; the failure of one section soon after tensioning might appear to be due to this high stress in addition to any fault in the jointing as suggested. It was stated that an I section had been designed originally, and it was not clear why a smaller beam with a bottom flange had not still been used when the original load was reduced. The writer found that a beam of identical area but 5 ft. deep instead of 6 ft. and with top flange $2\frac{1}{2}$ ft. wide instead of 3 ft., so that the remaining material could be placed in a lower flange 22 in. wide would have even lower stresses than the 2000 lb./sq. in. specified under the full range of live load, instead of a stress of 2657 lb./sq. in. given for the T-beam used.

With the I beam the initial stress would be the same as given in the paper, 2245 lb./sq. in., using the full initial cable tension, whereas it appeared that the effect of the increased initial cable tension had not been allowed for in the paper and the initial stress was therefore much higher. The cable pull for the I beam was identical with that for the T beam used and the writer

uld like the author's comments on this calculation ich was based on three simple equations which he d devised for the purpose of elucidating the best ape of beam for any circumstances.

Written Reply by the Author
Reply to Mr. Hajnal-Konyi and others

UPTURE MOMENT

In determining the resistance moment at rupture two ings must be known, the force and the moment arm. ie force may be assumed as being the aggregate resist-ce at rupture of the wires ; higher values have been und but cannot be depended upon. This force in the nsile flange will act as the centroid of the wires, since the wires will have been strained into the plastic ne. The line of action of the compressive force is less sy to determine, but it seems plausible with beams of ctions such as these to assume that it acts at the ntroid of the upper flange. In the case of the main am this gives a stress in the flange at rupture of oo lb. per sq. inch. This is high, but is acceptable nce (i) concrete of 6000 lb. per sq. in. at 28 days may ell attain this figure after, say, a year (ii) the *in situ* arapets on top of the main beam will certainly relieve e upper flange of stress at rupture—indeed it is not npossible that they should form an integral part of the am and greatly increase the moment arm. The gutter rbs may well act similarly in the secondary beams, ough here the flange stress at rupture is smaller and es not exceed 5100 lb. per sq. inch.

Such considerations give the following figures :—

) *Main beam*

upture tensile force = 3740 kips.
oment arm = 161.25 in.
hence Rupture moment = 6.03×10^5 kips in.
otal vertical design moment = 2.92×10^5 kips in.

. . Factor = 2.06

Subtracting the permanent loads not susceptible to ariation, we get the following factor on live loads :—

$$\frac{603-218}{292-218} = \frac{385}{74} = 5.2$$

) *Secondary beam*

upture tensile force = 730 kips.
oment arm = 56 in.
hence Rupture moment = 4.10×10^4 kips in.
otal vertical design moment = 1.93×10^4 kips in.

$$\therefore \text{ Factor } = \frac{4.1}{1.93} = 2.12$$

gain subtracting the permanent loads, we get the live ad factor :—

$$\frac{4.1-1.4}{0.532} = 5.08$$

It cannot be denied that there is an air of unreality out these calculations ; in beams of the size of the ain beam, it is difficult to see a load approaching this lculated rupture load without causing severe secondary fects of one sort and another ; some of these will be vourable, like the interaction of the beam with the perstructure above it, others will be unfavourable e the exaggeration of shear stresses due to torsion d the interaction of these effects is quite unpredictable, even under the simplest form of loading. Nevertheless, the calculation of the rupture load in bending was asked for ; for what it is worth, here it is.

Reply to Mr. Scott White

Enquiry has elicited the following replies to the two questions asked by Mr. Scott White.

(i) About one in twenty of the precast elements in the secondary beams were rejected due to various causes such as damage in handling, etc.

(ii) Approximately 70 incidents in stressing have been reported to date ; by "incident" is understood an occurrence necessitating restressing at a cone. Some five of these were due to broken wires, one wire being broken in a cable in each case. The remaining incidents were shared between the steel wedges flying off the jack during stressing and one or two wires pulling into the beam immediately after anchoring. This last type of incident occurred during a limited period of the works and appeared to be associated with the first batch of male cones cast in a newly set-up cone-making plant in which aluminium moulds were used for the first time ; after a few re-uses of these moulds the trouble seemed to disappear. In the event the defective male cones were removed and replaced by new ones. Several thousand stressing operations have been completed uneventfully on these works.

Reply to Prof. Robertson

The section proposed by Prof. Robertson for the secondary beam while being slightly heavier than that used certainly does all that he says it would ; the same cable force would suffice for this beam, using all eight cables at the maximum eccentricity at the centre of the span, i.e., giving each about 1 in. cover—an arrangement which has its difficulties but which is certainly possible using a similar horizontal movement of the cables in the bottom flange as is used in the main beam. Indeed, this section uses the concrete more effectively, with a better balanced stress distribution, than does that used in the structure ; it was in fact hinted in the course of the paper that a T-section had its disadvantages from this point of view.

Nevertheless, Professor Robertson's section would have had in the circumstances two rather serious drawbacks. In the first place, 30 in. breadth for the top flange is narrow for a 110 ft. span and the author would have hesitated before launching such a beam under its own weight without lateral support. In the second place, there would have been less latitude in arranging the cables to dodge the many holes needed for services.

It was these considerations, combined with a possibly illusory idea of simplifying formwork, that led to the employment of a T-section.

No figures were given in the paper for the initial stresses immediately after tensioning, for the reason that these stresses are extremely difficult, if not impossible, to estimate at all exactly. The prestressing of one beam takes time, during that time a large proportion of the creep of the steel and an appreciable proportion of the creep of the concrete has already taken place ; the larger part of the remainder takes place quite rapidly. In practical design a figure rather less than 15 per cent. is employed to check the stresses at the time of prestressing to ensure that they are not excessive.

The use of a high stress at this stage is permitted since it acts as an automatic test of the concrete ; this test in itself is a large factor of safety.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, December 18th, 1952, at 5 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership?

STUDENTS

CLARK, John Alan, of London.
COULL, Alexander, of Peterhead.
HEATON, Peter John, of Manchester.
HORSMAN, Peter, of Ripon, Yorks.
IRWIN, Keith Thompson, of London.
LEACH, Peter Edgar, of Manchester.
LEE, Jack, of Manchester.
MILES, Alan David, of London.
MISZEWSKI, Jan Stefan, of London.
NEWBY, Ronald William, of London.
PRYKE, John Fordham Speaight, B.A.Hons., Cantab., of Wormley, Herts.
RIMMER, William, of Burtonwood, Lancs.
RUMNEY, Derek Peter, of Stretford, Lancs.
RUSHTON, Basil James, of London.
SAXENA, Kailash Chandra, B.Sc. Agra, of Indore, India.
WHITE, Bryan John, of London.
YEO LIM HUI, of Singapore.

GRADUATES

ATHUKORALA, Don Gunasena, B.Sc.(Eng.) London, of London.
BENNETT-GOODMAN, John, of London.
BRILL, Herbert, B.Sc.(Eng.) Hons. London, of London.
CHITALE, Vaman Mahadeo, B.E.(Civil) Hons. Bombay, of Bombay, India.
D'LEMOS, Noel Brian, B.Sc.(Eng.) Aligarh, of Heston, Middlesex.
FEIT, Michael John, of London.
GALBRAITH, Roy, B.Sc.(Eng.) London, of London.
HALL, William Edwin, M.A. (Cantab.), of Blackpool, Lancs.
HAMER, Clifford, of London.
ISAACS, Basil, B.Sc.(Eng.) Rand, of London.
JACKSON, James Keith, B.Sc.(Eng.) London, D.I.C., of Preston, Lancs.
LANGE, Thomas, B.Sc.(Eng.) Cape Town, of Petersham, Surrey.
LYNDON, Alfred, B.Sc. Tech. Manchester, of Droylsden, Lancs.
MCDERMOTT, Alfred Stanley, M.A. Cantab., of Maidstone, Kent.
MACNALLY, Cormac Columbanus Peter, B.Sc.(Eng.) Belfast, of Belfast.
MADDOCKS, David Christopher, B.A. Cantab., of Claygate, Surrey.
OSS, Frederick Joseph, B.E. Bombay, of Poona Cantonment, India.
PRIOR, Geoffrey, B.Sc. Tech. Manchester, of Timperley, Cheshire.
ROBINSON, Douglas, B.Sc.(Eng.) Durham, of Fitzwilliam, Yorks.
ROOK, Harry, B.Sc.(Eng.) Glasgow, of Crawford Lanarkshire.
SPURWAY, Ronald Victor, B.Sc.(Eng.) London, of Harrow, Middlesex.
THRELFALL, Robert Malcolm, B.Sc. Tech. Hons. Manchester, of Manchester.
TURNER, John Roger, of London.
WESTALL, John Butterworth, B.Sc. Tech. Hons. Manchester, of Manchester.
WILSON, Bernard Arthur, B.Sc.(Eng.) Hons. London, of Portsmouth.

ASSOCIATE

PRICE, Henry Wynter, of Pretoria, South Africa.

MEMBER

GOURLAY, James Fleming, B.Sc.(Eng.) Glasgow, A.M.I.C.E., of Glasgow.

TRANSFERS

Students to Graduates

ALCOCK, Bruce Lawrence, of Port Elizabeth, South Africa.
BURMAN, David Charles, of Liverpool.
EVANS, Clifford John, B.A. Hons. Cantab., of Penarth, Glam.
HOWDEN, Ivor, B.Sc.(Eng.) London, of Norton-on-Tees.
JONES, Malcolm, of Batley, Yorks.

Graduates to Associate-Members

BOUSTANY, Fiaz Wadie, B.Sc.(Eng.) Hons. London, of Kuwait, Persian Gulf.
HAMP, Eric James, of London.
HILL, David Alexander Howard, B.Sc.(Eng.), Cape Town, of Pretoria, South Africa.
HOGAN, Eugene Peter, of Crosby, Liverpool 23.
LEVY, Joseph, of Glasgow, S.2.
LOVEJOY, Edward Geoffrey, of Anglesey.
SEVILLE, Stephen Wyatt, of Manchester.
WASHINGTON, John Brian, of London.

Associate-Members to Members

LAKE, John Owen Arthur, of London.
SHAHANI, Chetanram Mangharam, B.Sc.(Civil) Edinburgh, A.M.I.C.E., of Calcutta, India.

Members to Retired Members

BADIAN, Avner, of Haifa, Israel.
BARKER, Theodore Delabere, M.I.Mech.E., of Christchurch, New Zealand.
COTTERELL, Ernest Lawrence, of Wolverhampton.
GALBRAITH, William, of Bexhill-on-Sea.
IVINSON, Frank, of Stockport, Lancs.
KNIGHT, Bernard Howard, D.Sc., Ph.D., M.I.C.E., F.R.I.C.S., of Wraysbury, Bucks.
MAINWARING, Harry, M.C., of Altrincham, Cheshire.
MONCRIEFF, Lt. Col. Alan, O.B.E., M.C., M.A.(Cantab.), A.M.I.C.E., of Wadhurst, Sussex.
OKELL, Robert, M.C., B.Sc., M.I.Mech.E., A.M.I.C.E., of Durban, South Africa.
OWEN, Sydney, of Widnes, Lancs.
PITT, Albert Ovenden, of Durban, South Africa.
WOLSTENHOLME, Walter, M.I.Mech.E., of Johannesburg, South Africa.

Re-Admission

Associate-Membership

ICKELL, Richard Goulden, M.I.C.E., of Wellington, New Zealand.

OBITUARY

The Council regret to announce the deaths of JOHN LLIAM BARROW, LEON EMILE CALLEBAUT (Members); ofessor FREDERICK CHARLES LEA, ALBERT STANLEY ENCER (Retired Members); LESLIE ALLEN, JAMES FF, HARRY STUART MORGAN, ALFRED EDWARD LLIAMS (Associate-Members).

RESIGNATIONS

Notification was given that the Council had accepted th regret the resignations of JOHN ALFRED DAVENRT, CHARLES REGINALD FORD, HENRY GARDNER, ONEL DOUGLAS LEONARD, MAURICE SPENCER, CHARLES ERBERT ASLIN (Members); HERBERT DUNCAN BINDY (Retired Member); HERBERT HENRY CLARK, IFFORD BROADBENT DEAN, FREDERICK ROBERT YHILL, HAROLD WILLIAM SHARPE (Associates); ORGE CROSSLEY, JOSEPH GIMPEL, ROBERT RUSSELL GLIS, AUSTIN JOHN MORGAN JONES, WILLIAM ARCHILD MORRIS, HARRY NURSE, JOHN LLOYD ROACH, ALTER GEORGE WHINCOP (Associate-Members); LOUIS INTERTON ACKROYD, ARTHUR LAWRENCE HORNIBROOK YLIS, DUNCAN MACGILLIVRAY BEATON, RONALD ORGE COX, DATTATRAY VAMAN DESHPANDE, JOHN RTRAM DONAT, ERIC ARTHUR SINCLAIR GUY, HUGH WARD HUTCHINSON, CHARLES JACK KIMBER, JOHN DERICK LAWSON, ALAN PAUL SENIOR, GEORGE VID SPEARING, RAYMOND TWOSE, JOHN MICHAEL ALSH, JOHN PURCHASE WILKINS (Graduates); ILLIP ARTHUR COLLINS, PETER HOWELL DAVIES, ALTER BISHOP DAVIES, LIONEL FRANCIS SELEM DRAKE, FRED WALLACE GOULD, DAVID SYDNEY MACKAY ERR, JOHN DEREK MARSHALL, PETER JOHN TANNER, MES WILLIAM HENRY TAYLOR, PETER IAN WOOD LKERS, EDWIN JOHN WASDELL, CHARLIE ANTONY ADDRINGTON WOODS (Students).

EXAMINATIONS—JULY, 1953

The Examinations of the Institution will next be held centres in the United Kingdom and overseas on ly 14th and 15th, 1953 (Graduateship) and July 16th d 17th (Associate-Membership).

EXAMINATIONS

EPARATION FOR THE EXAMINATIONS OF THE INSTITUON BY ATTENDANCE AT TECHNICAL COLLEGES

A candidate for Graduateship or Associate-Membership y be able to attend a technical college; these notes e intended to guide him in choosing the most suitable truction.

EPARATION FOR THE GRADUATESHIP EXAMINATION

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National plomas in Building or Engineering.

(b) Part-time day or evening courses for Higher tional Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma mplying with Appendix II, Section V, of the Regulans Governing Admission to Membership, the candidate ll be exempted from the Graduateship Examination. Alternatively, he may study subjects selected from e available courses and sit the Graduateship Examinan. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

PREPARATION FOR THE ASSOCIATE-MEMBERSHIP EXAMINATION

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in the first category provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

The Colleges which have informed the Institution that courses in Structural Engineering are available are :—

Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts, and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Salford Royal Technical College.
South-West Essex Technical College, Walthamstow, E.17.
Stockport College for Further Education.
Willesden Technical College, N.W.10.

Colleges in the second category provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete. The colleges which have informed the Institution that such courses are available are :—

Bridgend Technical College.
Brighton Technical College.
Cardiff Technical College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.

Students attending colleges in the first category are advised to take the organised courses in Structural

Engineering. Students of Graduate Membership standard will usually be allowed to select subjects from courses provided by colleges in the second category.

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, February 12th, 1953

Ordinary Meeting at 6 p.m., when Dr. F. G. Thomas, M.I.C.E. (Member of Council), will give a paper on "The Strength of Brickwork."

Thursday, February 26th, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. P. L. Capper, T.D., M.Sc., A.M.I.C.E. (Member of Council), will give a paper on "Soil Mechanics in relation to Structural Engineering."

Thursday, March 12th, 1953

Joint Meeting with the Institute of Welding, at 6 p.m., when Mr. E. M. Lewis will give a paper on "The Construction of the new Testing Laboratory of the British Welding Research Association at Abington."

Thursday, March 26th, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Professor A. G. Pugsley, O.B.E., F.R.S., D.Sc., M.I.C.E., F.R.Ae.S. (Vice-President), will give a paper on "A Simple Theory of Suspension Bridges."

Thursday, April 23rd, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. J. H. A. Crockett, B.Sc., A.C.G.I., D.I.C., A.M.I.C.E. (Associate-Member), and Mr. D. B. O'Neill, B.Sc., will give a paper on "Modern Machine Foundation Technique."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

JOURNAL CASES AND BINDING, 1952

A binding case can be supplied for the twelve issues of the Journal, January-December, 1952 (Vol. 30), price 11/6d. post free. The price for binding volumes is 26/6d. per volume, inclusive. This is for the half-leather binding which has been in use for some years.

It is requested that all parcels and Journals forwarded for binding should bear the name, address and rank of the member concerned. All volumes for binding must be despatched to the Institution by March 31st, 1953.

An Index will be included in all volumes bound. This Index will not be generally distributed, but members and others wishing to have a copy should apply to the Secretary.

DRURY MEDAL AWARD

The fourth competition for the above award will take place in 1953. The subject is the design of the structure of a new factory building. The material of construction is entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of structural arrangement. Economy in the use of steel is an important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary; envelopes to be marked in the top left-hand corner, "Drury Medal Award."

The closing date for the competition is October 1st, 1953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say, primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution, or failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

MACLACHLAN LECTURE COMPETITION, 1953

The closing date for the receipt of entries for the next MacLachlan Lecture Competition will be Tuesday, March 31st, 1953. Particulars of the Competition are as follows :—

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering so long as in every second year the subject shall be confined to steel structures. (1953 is one of these years.)

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s. 0d.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer these sums to the Research Fund of the Institution.

PAPERS FOR PUBLICATION

The Literature Committee have under consideration the selection of papers for inclusion in the Sessional Programme for 1953-1954. Members who may wish to offer papers during the coming Session are invited to communicate with the Secretary.

The Committee would also be glad to consider offers papers for publication without discussion at a meeting.

The following is a summary of the Committee's quirements relating to articles and papers ; a copy the full conditions may be obtained from the Secretary.

1) Articles must be of an appropriate character, ving a bearing upon structural engineering or upon ne kindred scientific or constructional subject, and ist be approved by the Literature Committee. A ort title is an advantage.

(2) Contributions must be original either in subject-tter or in presentation. Articles which have already en published or have been read to other organised dies, or are carelessly prepared, will not be accepted publication.

(3) The style of writing will necessarily vary with the dividual, but authors are requested to write as plainly d simply as their subject will allow. Papers should written in the third person.

(4) Where the subject allows, a brief introduction or nopsis should state clearly the purpose and scope of e paper or article, and the author's conclusions or commendations should be summarised at the end of e paper.

In order to facilitate the indexing of articles for refer-ce, the author will be required in addition to prepare a ort precis not exceeding 25 words for inclusion under e title of the paper on the contents page of the Journal.

(5) Illustrations are desirable where they assist in plaining the context or are fundamental to the bject. They should not be used if unnecessary for ese purposes. Illustrations may be either line draw-gs or photographs.

(6) Line drawings must be specially prepared for production on smooth white paper or clear tracing per, with heavy main lines and large clear lettering awn in indian ink with a mapping pen. Alternatively, e author may submit drawings on one sheet of paper th the relevant lettering on a cover sheet of tracing per.

The printed page of THE STRUCTURAL ENGINEER is in. wide by 10 in. deep. The drawings, where prac-cable, should be prepared not larger than twice this ze with a view to half-scale reproduction. Unavoidably rge drawings which require reduction to one-third size less, must be specially heavy and with proportionately rge lettering for clear reproduction. Ordinary working awings are not satisfactory.

(7) Where photographs are submitted they should be inted *black on glossy paper.*

(8) MS. typewritten in double spacing should be bmitted in duplicate.

Brevity is an advantage and papers should not rmally exceed 7,500 words in length.

THE LATE MR. C. F. PRIESTLEY

Members of the Institution who were acquainted with r. C. F. Priestley, a former Member, will regret to learn at he died at Parramatta General Hospital, Australia, November 13th, 1952, after a heart seizure.

ONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held at , Upper Belgrave Street, London, S.W.1, on Tuesday, bruary 24th, 1953, at 6 p.m., when Lt.-Colonel G. W. irkland, M.B.E. (Member of Council), will give a per on " Prestressed Concrete."

Hon. Secretary : C. Allen Brown, 43, Coolgardie venue, Highams Park, London, E.4.

ADDITIONS TO THE LIBRARY

British Constructional Steelwork Association. Publication No. 5—*The Collapse Method of Design—being the application of the Plastic Theory to the Design of Mild Steel Beams and Rigid Frames.* London, 1952.

British Iron and Steel Research Association Corrosion Committee : *The Fight against Rust.* London, 1952.

Institution of Mechanical Engineers, *Proceedings of Automobile Division,* 1950-51.

International Association for Bridge and Structural Engineering Publications, Vol. 11, 1951 and Vol. 12, 1952. Zurich, 1952.

International Association for Bridge and Structural Engineering, Fourth Congress, 1952, *Preliminary Publication.* Presented by Major R. F. Maitland.

KNIGHT, B. H., and KNIGHT, R. G. *Builders' Materials.* London, 1948. Presented by Mr. A. T. Wadi Williams

National Bureau of Standards Circular No. 520 : *Mechanical Properties of Metals at Low Temperatures.* Washington, 1952.

SOWERS, G. B., and SOWERS, G. F. *Introductory Soil Mechanics and Foundations.* New York, 1951. Presented by Mr. C. B. Brown.

Smithsonian Institution Annual Report, 1950. Washington, 1951.

TOFT, L., and MCKAY, A. D. D. *Practical Mathematics,* Vol. II. 3rd Edn. London, 1952.

WHITE PARSONS, G. D. *Callender-Hamilton Bridge Handbook : Highway Bridge Type " B."* London, 1952.

CORRECTION

" THE ASSUMED DEFLECTION METHOD FOR DETERMINING TRANSVERSE STRESSES IN SLABS SUPPORTED ON TWO SIDES "

In the precis of the above Article published in THE STRUCTURAL ENGINEER of August, 1952, the term I_L was used to describe " moment of inertia per unit width of slab."

This must be calculated in the following manner :—

Let I_B be the moment of inertia of the slab section, or of a fixed width of the slab section about the transverse axis.

Let I_R be the moment of inertia of the enclosing rectangle of dimensions " B " width and " D " depth.

Then the ratio $\frac{I_B}{I_R}$ is termed the " shape factor " and is given the symbol α, so that

$$I_B = \alpha I_R = \frac{\alpha B D^3}{12}$$

I_L is then given by $\frac{I_B}{B}$, so that

$$I_L = \frac{\alpha D^3}{12}$$

and this must be used in the equations.

The dimensions of I_T are unaffected, and it can be calculated in the usual fashion, the design moment will then be given for the appropriate linear unit of slab surface.

The author regrets that the original statement may have been misleading and trusts that no inconvenience has been caused through the use of the equations as at first published.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Tuesday, February 3rd, 1953

Annual Dinner and Dance, Longford Hall, Stretford. 6.45 p.m. for 7.15 p.m.

Wednesday, February 11th, 1953

Mr. G. A. Gardner, O.B.E., (Member) on "The Structural Use of Steel in Building" at the College of Technology, Liverpool, 7 p.m.

Tuesday, February 24th, 1953

Mr. P. W. Rowe, B.Sc., Ph.D., A.M.I.C.E., on "Developments in the Design of Sheet Pile Walls."

Tuesday, March 3rd, 1953

Joint Meeting with Reinforced Concrete Association. Mr. E. Shepley, B.Sc., A.M.I.C.E., on "Prestressed Concrete Framework for Liverpool University Medical School."

Thursday, March 26th, 1953

Joint Meeting with the Institution of Civil Engineers, North-Western Association.

All meetings, unless otherwise stated, will be held in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, February 17th, 1953

Mr. O. W. Jones, B.Sc., A.M.I.C.E. (Member), on "Reinforced Concrete Foundations and Structures for Blast Furnaces and Materials and Handling Plant at Shotton, Nr. Chester," at King's Hall, Queen Street, Derby, at 7.0 p.m.

Friday, February 27th, 1953

Mr. N. T. Grant, on "Experiences with Concrete."

Friday, March 27th, 1953

Details to be announced.

Friday, April 24th, 1953

Annual General Meeting. Followed by paper on "Reinforced Concrete Foundations and Structures for Blast Furnaces and Materials and Handling Plant at Shotton, Nr. Chester," by Mr. O. W. Jones, B.Sc., A.M.I.C.E. (Member).

All meetings, unless otherwise stated, will be held at the James Watt Memorial Institute, Birmingham, at 6.0 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E. 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES BRANCH—GRADUATES' AND STUDENTS' SECTION

The following meeting has been arranged :—

Tuesday, March 31st, 1953

Annual General Meeting at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7.0 p.m.

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, February 3rd, 1953

Mr. B. A. E. Hiley, M.I.C.E. (Member of Council), on "Electricity Generating Stations," at Middlesbrough.

Wednesday, February 4th, 1953

The above meeting will be repeated at Newcastle.

Tuesday, March 3rd, 1953

Ladies' Guest Night, at Middlesbrough.

Wednesday, March 4th, 1953

Ladies' Guest Night at Newcastle.

Tuesday, March 31st, 1953

Annual General Meeting.

All meetings will commence at 6.30 p.m., the Middlesbrough meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and those at Newcastle in the Neville Hall, near Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, February 10th, 1953

Annual Dinner and Social Function. Visit of the President and Secretary of the Institution, at Grand Central Hotel, Royal Avenue, Belfast, at 6.30 p.m.

Tuesday, March 3rd, 1953

Mr. J. C. Malcolmson, B.Sc. (Member), on "Recent Developments in Prefabricated Concrete Structures."

Tuesday, March 31st, 1953

Annual General Meeting.

All meetings, unless otherwise stated, will be held in the College of Technology, Belfast, at 6.45 p.m., preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6.0 p.m.

Hon. Secretary : S. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, January 20th, 1953

Mr. C. H. Hill, on "The Design and Erection of Television Masts," at Royal Circus Hotel, Royal Circus, Edinburgh, 3, at 6.0 p.m.

Friday, February 6th, 1953

Mr. A. Frood, Mr. Peter Murray, B.Sc., M.I.C.E., and Mr. James Robertson, B.Sc., M.I.C.E., on "Problem Jobs—Contractors' Solutions." Joint Meeting with the Glasgow and West of Scotland Association of the Institution of Civil Engineers at the Institute of Engineers and Shipbuilders, Elmbank Street, Glasgow, at 7.15 p.m.

Wednesday, March 18th, 1953

Mr. H. Nelson, B.Sc., A.R.T.C., on "Tension Members in Engineering Structures," at the Royal Technical College, George Street, Glasgow, at 6.0 p.m.

Tuesday, April 14th, 1953

Annual General Meeting. At Ca'doro Restaurant, Union Street, Glasgow, at 6.0 p.m.

Hon. Secretary : D. G. Drummond, B.Sc., M.I.Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

The following meeting has been arranged :—

Friday, February 6th, 1953

Mr. F. R. Bullen, B.Sc., M.I.C.E. (Hon. Curator), on Unusual Design for a Large Constructional Shop," at Duke of Cornwall Hotel, Millbay, Plymouth, at p.m.

Hon. Secretary : E. W. Howells, M.I.Struct.E., Messrs. T. L. Harding & Sons, Ltd., 10-12, Market eet, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, February 11th, 1953

Mr. D. Manolopoulos (Member), on " Report on the Congress of the International Association of Bridge d Structural Engineering," at Swansea.

Tuesday, February 17th, 1953

The above meeting will be repeated at Cardiff.

Wednesday, March 11th, 1953

Mr. J. R. M. McEwen, B.Sc., on " Tubular Strucres," at Swansea.

Tuesday, March 17th, 1953

The above meeting will be repeated at Cardiff.

Friday, March 27th, 1953

Branch Annual Dinner at Swansea.

Tuesday, April 14th, 1953

Mr. J. R. M. Poole, on " Recent Developments in nber Structures."

Meetings at Cardiff will be held at the South Wales stitute of Engineers, Park Place, at 6.30 p.m.

Meetings in Swansea will be held at the Mackworth tel at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

The second meeting of the Session was a Combined eting with the South-Western Association of the stitution of Civil Engineers, held in the Geology cture Theatre of the University of Bristol, on Friday, vember 6th, 1952.

Mr. G. P. Bridges, A.M.I.C.E. (Member), presented paper entitled " The Design and Construction of inforced Concrete Silos and Bunkers," revealing his pert knowledge of both the theoretical and practical ues. Mr. Bridges gave excellent replies to the many estions put to him by members and friends present.

A Vote of Thanks was proposed by Mr. N. S. Cox, Sc., M.I.C.E.

The following meetings have been arranged :—

Friday, February 6th, 1953

Mr. F. G. Clarke (Associate Member), on " Some Local ntracts and Welded Steelwork for a Bus Garage."

Wednesday, February 18th, 1953

Annual Dinner.

Thursday, March 5th, 1953

Combined Meeting with the Institution of Civil gineers. Mr. L. Richardson, A.M.I.C.E. (Associate Member), on " Plymouth ' B ' Power Station," at the Reception Room, University of Bristol, at 5.30 p.m. Tea at 5.0 p.m.

Friday, April 10th, 1953

Annual General Meeting. Followed by Film Show and/or Papers presented by Junior Members.

All meetings, unless otherwise stated, will be held in the University of Bristol Geology Lecture Theatre (entrance University Road), at 6.0 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 39, Effingham Road, St. Andrew's Park, Bristol, 6.

YORKSHIRE BRANCH

The second meeting of the Yorkshire Branch was held on Wednesday, November 19th, in the University, Leeds, when a paper entitled " Concrete Grain Silos at Louth," was given by Mr. G. C. Cummings, and was illustrated with a large number of slides. A Vote of Thanks to the speaker was proposed by Dr. S. Champion, and seconded by Mr. A. Robb, and was carried with acclamation.

A Joint Meeting of the Yorkshire Branch and the Yorkshire Association of the Institution of Civil Engineers was held on Thursday, November 27th, at the Blue Bell Hotel, Scunthorpe, when about 45 members and visitors assembled to hear a paper on " The Plastic Theory and its Application to the Design of Mild Steel Beams and Rigid Frames," by Mr. F. A. Partridge, B.Sc.(Eng.), M.I.C.E., A.C.G.I., M.Inst.W. The paper was well illustrated and provoked keen discussion.

The following meetings have been arranged :—

Wednesday, February 18th, 1953

Dr. S. Mackey, M.E., B.Sc., A.M.I.C.E.I. (Associate-Member), on " Secondary Stresses in Steel Bridge Girders."

Wednesday, March 18th, 1953

Mr. H. E. Manning, B.Sc., M.I.C.E. (Member of Council), on " Report on the Papers given on Concrete Shell Roofs in London, July, 1952."

Friday, April 17th, 1953

Annual Dinner and Dance, at the Parkway Hotel, Bramhope, Leeds, at 7.0 p.m.

Wednesday, April 22nd, 1953

Annual General Meeting. Followed by " Sea Defence Works," by Mr. J. Dossor, M.I.C.E., A.M.I.Mech.E. (Member).

Unless otherwise stated, all meetings will be held at The University, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. Phone : 34-1111 Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

Book Reviews

System of Tables for Quick and Accurate Solving of any Continuous Beam (2nd Edition, in English), by A. P. Skayannis (London : Lange, Maxwell & Springer, 1949). 9¾ in. × 6¾ in. 43 pp., plus diagram. 21s.

These tables will be of value to all those who deal with the problems of continuous beams as they give a rapid and accurate solution for computing the distances of the fixed points and eliminate the need for graphical construction.

In addition to the tables, examples are given together with their solutions, and the volume includes a nomographic chart for the determination of the positions of the fixed points.

A Measuring Diagram for Daylight Illumination for the Measurement, Predetermination and Representation of Natural Lighting, by Percy J. Waldram (London : B. T. Batsford, 1950). 19 pp. 9¾ in. × 6 in., 7 figs. and chart. 5s.

This booklet, for the use of architects and surveyors, contains copies of measuring diagrams for the solution of problems on the natural lighting of interiors. Clear instructions are given and a large-scale chart is provided.

Reinforced Concrete, by Oscar Faber. (London : E. & F. Spon, Ltd., 1952.) 8¾ in. × 5½ in., 232 pp. 30s.

This is a new presentation of well-established principles and methods, mainly in regard to the design of reinforced concrete beams, columns and slabs, as recommended by the 1948 Code of Practice which was produced under the Chairmanship of the author.

It is written in a clear and simple style and is a valuable exposition of design technique supported by references to many years of practical experience. It could be read and appreciated by a student having quite an elementary knowledge of strength of materials. The author has commendably attacked pretence to accuracy by mathematical display, but it might have been wiser to have recommended to the inexperienced, discrimination in regard to accuracy, as some readers may have the impression that reinforced concrete design calculations are always based on very vague assumptions. For instance, the Factor of Safety in regard to failure by steel yield can be computed fairly accurately in a reinforced concrete beam, but in regard to failure by concrete crushing only within wide limits. Also, it is important for students to appreciate the difference between calculations which are based on well-established safe limits and those which are just approximate.

The design of shell structures and prestressed concrete have now developed so as to require the attention of specialists. These subjects are introduced, but as the author indicates, they hardly come within the scope of the book. The excellent frontispiece showing Esbly Bridge is a fitting tribute to the work of Freyssinet but it is a pity that in regard to shell construction the pioneer analytical work which has been published during the last twenty years, and which is of historic importance, is given scant recognition.

A useful chapter on chimney design is included.

A demonstration is given on page 24 which shows clearly that the cost of reinforced concrete building frameworks and the quantity of steel required is less than for structural steelwork. This is particularly appropriate at the present time.

Writing a technical textbook is an arduous task, and it is greatly to the credit of a busy consultant such as Dr. Faber that he has been able to find the time to produce this useful book.

A. A. L. B.

The Resistance of Piles to Penetration, by Russell V. Allin, M.I.C.E. (London: Spon: 2nd edition 1951.) 30 pp. 67 Tables. 5¾ in. × 9½ in. 21s.

Having read the first 30 pages of this book, with the greatest interest, and much profit, one turns over, eager for more of the same quality, only to find in the remaining 93 pages an arid waste of tables. The Author states in the preface that " these tables have received some criticism on account of their unavoidable suggestion of an accuracy, which it is well known to practical engineers no piling formulæ can achieve," and he is at some pains to find justification for their inclusion. The claim that " they show the quantitative effects, on ultimate resistance, of varying the driving conditions, sizes and lengths" is hardly sufficient, since the engineer who wishes to analyse those effects will adopt a more elegant expedient than the extraction of data from tables of "approximate values based on average soil conditions." A few pages describing analytical methods, and other aids to the use of the formula, would have avoided the necessity of giving tables covering (but only for average conditions) the whole range from 12 in. × 12 in. timber piles driven with a ½-ton hammer, to 24 in. × 24 in. reinforced concrete piles driven with a 10-ton hammer. It is only right to say that these tables appear to constitute the Author's main purpose, but it is a purpose he might well have abandoned, using the space so saved for a continuation of his excellent exposition in the preceding chapters.

G. B. R. P.

Means of Escape in Case of Fire, by Horace J. Brend, A.R.I.C.S. (London : Pitman, 1952.) 155 pp. 5½ in. × 8¾in. 15s.

A treatise on means of escape by Mr. Horace Brend is something which carries with it authority since there can be few people who have had greater experience in the provision of means of escape than Mr. Brend, who for many years has been one of the senior officials in the Architect's Department of the London County Council dealing with this aspect of buildings and their occupation. It is made clear in the introduction that the opinions expressed in the book are the writer's own and, as such, should not in any manner implicate the Council or its officers.

The book is divided into two parts, the first giving a general precis of the several Acts which call for certain standards of escape and protection. These include the London Building Acts (Amendment) Act, 1939, Factories Act, 1937, London County Council (Celluloid) Act, 1915, Theatres Act, 1843 and Related Legislation, Public Health Act, 1936, and the Celluloid and Cinematograph Film Act, 1922. It is most useful to have the relative sections of these Acts grouped for easy reference in a single volume. The second part sets out the principles of means of escape applied to planning, together with numerous illustrations indicating typical arrangements.

The whole is compressed into 150 pages, providing a most valuable and handy size reference on this important aspect of planning and construction.

A. H. L.

Simple Theory of Suspension Bridges*

By Professor A. G. Pugsley, O.B.E., D.Sc., F.R.S.

S1 Introduction

In spite of the growing importance of suspension dges for long spans, English text-books on the theory structures commonly either make no reference to spension bridges or else give only a brief account Rankine's approximate theory.[1] In the latter case is usual to outline the method as applied to a bridge th a three-hinged stiffening girder, to mention that e results so obtained overestimate the bending actions the girder, and perhaps to indicate that, with still s accuracy, the method can be applied to the case of a o-pinned girder. If anything is said of the "deflec-on theory" in common use in the U.S.A., and outlined some American text-books[2], [3], it is usual only to mark on its greater accuracy without giving much dication of its essential nature and limitations.

This paucity of English material on the subject is w more apparent than real. Some fifteen years ago uthwell showed how relaxation methods could be plied to a suspension bridge, and worked out with kinson[4] a numerical example in which allowances ere made for some factors usually ignored by American orkers. More recently this method has been applied, th further detailed improvement, by Crosthwaite[5] the course of working on the design of the proposed vern bridge. More recently still, the writer has tlined a method of treatment based on the use of xibility coefficients to represent the behaviour of th the cable and the stiffening girder[6], and the accurate lculation of these coefficients has been discussed by arkland[7].

Thus for the analysis of a particular bridge under a rticular system of loading there are now available at ast three alternative methods of approach—the fferential equation treatment of the deflection theory, e successive approximation treatment of the relaxation eory, and the simultaneous linear equation approach the flexibility coefficient method. But none of these proved and more accurate methods make for ready sign generalisations, and although they represent ccessive improvements in the physical understanding the behaviour of suspension bridges, none have the nplicity and the design appeal of the early theory of ankine.

It was with this background in mind, and in the urse of working on the development of the flexibility efficient approach, that the writer conceived the mple approximate theory of the present paper. Essen-lly a suspension bridge consists of two structures—a avy cable and a beam—both of which are separately pable of carrying transverse loads, but with very fferent deflections, and their joint behaviour must rgely depend upon their relative stiffness under ansverse loads. Rankine assumed that in practice is relative stiffness would be such that no changes of ape in the cable form would occur ; but are practical idges really in this category ? And, if not, what are e effects of changes of relative stiffness ? These are the ain questions to which a simple theory—if it is to prove on Rankine's method—must provide an answer. And the questions are not just academic ones, for it is already well known that Rankine's method gives results for the bending moments on practical suspension bridge girders that are sometimes at least twice the probable values[2].

In the first section of this paper the place of the relative stiffness of beam and cable, and the natural mode of its expression, are examined by dimensional theory. Thereafter, an analogy is drawn between a beam supported by a cable and a beam supported by an elastic foundation, and a whole field of simple analysis for the latter thereby made available for the former ; the stiffness parameter of dimensional theory is thus brought to effective use. This fact is then illustrated by applying the method to examine a typical bridge, first as regards the girder bending moment problem upon which attention has been primarily concentrated, and then as regards the tensions in the cable and suspension rods. A final section examines briefly the relevance of the method to the prediction of the natural frequencies of a suspension bridge in flexure.

S2 Dimensional Theory

It will help to clarify the problem and to coordinate the basic factors involved to look at it first in terms of dimensional theory. For this purpose we have to decide upon the main parameters concerned. To do this, let us assume that our centre of interest is the bending moment on a stiffening girder at a given point along its span when the bridge is under a given system of loading. To simplify our statement of the loading, consider a single transverse load P somewhere on the bridge (so long as the deflections are small, the behaviour of the bridge will be linear and the effects of any more general system of loading can be analysed by the principle of superposition) and assume that the bridge has just a single span with cables that are effectively fixed in space at the tops of the towers. This will enable us to isolate the essential problems of a suspension bridge without involving the complications—not fundamental ones—arising from flexibility of towers or from extra spans.

So far as the stiffening girder is concerned, were the cable and suspension rods not present the bending moment upon it would be a simple function of the load P and the span L ; but the presence of the cable and rods introduces some partition of the load between cable and girder, and so the effective stiffness EI of the girder must be an important parameter. The shear stiffness of the girder may also play some part, but let us assume that we can allow for this adequately by a suitable evaluation of EI.

The cable on its own would carry the load P largely by a change of its shape rather than by any effect of its own small bending stiffness or of its slight increase of length. Its resistance to the load would thus arise primarily from the fact that the application of P would result in a net lifting of the cable, so doing work against gravity[8]. This resistance due to gravity is governed entirely by the weight of the cable per unit length measured along the span, the span L and the dip d of the cable. But the bridge decking hangs from the cable and if we neglect the extensibility of the vertical rods between cable and girder, this gravity stiffness of

*Paper to be read before the Institution of Structural Engineers 11, Upper Belgrave Street, London, S.W.1, on Thursday, March th, 1953, at 6 p.m.

the cable will depend on w, the weight of the whole bridge per unit span.

Thus we have as the main parameters governing the bending moment due to the load :

$$P,\ L,\ EI,\ w,\ d,$$

and may write $M = \phi(P, L, EI, w, d)$

Examination of this function by the method of dimensions leads to the following expression as a typical term in any expansion representing it

$$M = k' \cdot \frac{EI}{L} \cdot \left(\frac{wL^3}{EI}\right)^p \cdot \left(\frac{d}{L}\right)^q \cdot \left(\frac{PL^2}{EI}\right)^r$$

The non-dimensional terms here can be raised to any power and multiplied one by the other without effecting the homogeneity of the expression. Utilising this freedom to alter the grouping towards physically significant terms, we can write that in general

$$M = PL\Psi\left\{\frac{EId}{wL^4},\ \frac{L}{d},\ \frac{PL^3}{EId}\right\} \quad \ldots\ldots\ldots (1)$$

Of the parameters in this function, $\frac{EId}{wL^4}$ is identifiable as kindred to the stiffness factor $S' = \left(\frac{8EId}{wL^4}\right)^{\frac{1}{2}}$ used by Steinman and Baker [2], and as proportional to the ratio $R = \frac{24EId}{wL^4}$ previously used by the writer[9] for the ratio of the beam and cable stiffnesses. The shape factor $\frac{L}{d}$, within the small variations of $\frac{L}{d}$ adopted in practice, is known from deflection and flexibility theory calculations to be unimportant. The factor* $\frac{PL^3}{EId}$ expresses a ratio of girder deflection to cable dip and for small deflections would be expected to have negligible effect ; moreover, omission of this term is clearly consistent with the assumption of linear response to the load P. There are thus good grounds for believing that our general expression for M may with sufficient accuracy in many cases be reduced to the form

$$M = PL\ \phi\left(\frac{EId}{wL^4}\right) \quad \ldots\ldots\ldots (2)$$

*Multiplication of this factor by the first in (1) gives $\frac{P}{wL}$, the ratio of the applied load to the weight of the cable, as an interesting alternative form.

S3 Elastic Foundation Analogy

It is clear from the foregoing paragraph that any substantial improvement, in simple terms, of Rankine's theory depends upon the introduction into the analysis of factors representing the stiffness ratio developed in

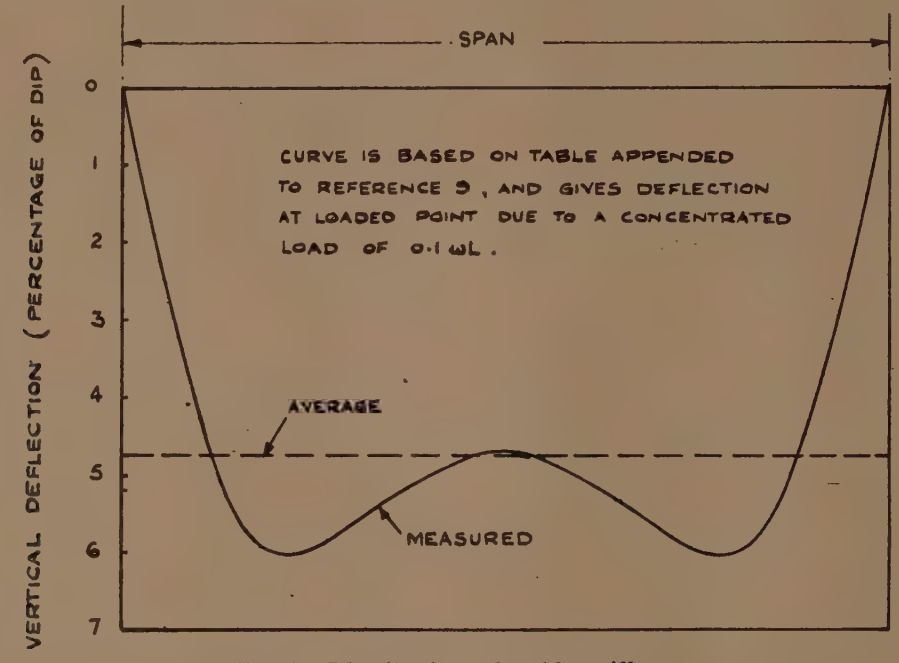

Fig. 1.—Distribution of cable stiffness

dimensional form in (2). Now a relative stiffness problem well known to structural engineers and having a fully developed literature of its own is that concerning, under various conditions, the behaviour of a beam resting on an idealised elastic foundation, and it occurred to the writer that advantage might be taken of this for the treatment of the suspension bridge problem.

In the first place, the stiffening girders of suspension bridges are more or less uniform structures supported in effect continuously along their lengths by the suspension cables above. And in the second place, the cables provide almost uniform resistance to vertical displacement along the greater part of their length ; that this is so is illustrated by Fig. 1 and had already been observed experimentally by the author[9] and is confirmed by the theoretical results given by Markland[7]. All that seemed necessary, therefore, once the analogy was appreciated, was to determine how best to relate the foundation stiffness (governed by its modulus k) to the stiffness of an actual cable.

Happily, as is usual in other fields of application of elastic foundation theory, the results sought—in our case primarily the bending moments on stiffening girders—are markedly insensitive to errors in k. For a long beam resting on an elastic foundation and carrying a concentrated load, the bending moment under the load is inversely proportional to the 4th root of k*, so that a

*See equation (9), p. 78.

10 per cent. error in k may result in only some 10 per cent. error in moment.

The stiffness curve of Fig. 1 is based upon deflection measurements at nine stations evenly spaced across the cable span, the cable having a dip to span ratio of 1 to 10, a ratio representative of practice. It will be seen that the average deflection at each station, due to a local load of $0.1wL$, is 4.76 per cent. of the cable dip. Thus if we are to replace the actual cable by an elastic foundation of uniform properties, we can conceive of the foundation, in this case, as consisting of nine independent vertical springs each of average stiffness. Under such conditions, by applying a load of $0.1wL$ to each spring, the whole foundation would move down an amount equal to $\frac{4.76}{100}d$. The total load effecting this would be $0.9\,wL$ spread over a length L. Thus the modulus of the foundation would be

$$k = \frac{0.9wL}{L \times .0476d} = 18.9\,\frac{w}{d}.$$

This gives us at once the form and the order of the equivalent modulus required.

It is clear, however, that as in the comparable railway track foundation problem the precise magnitude to be given to k must be decided empirically; and many would naturally place most emphasis on a value found to give the most accurate predictions in practice for girder bending moments. At this stage, therefore, there is no point in extreme precision, both on account of lack of extensive experience and on the knowledge that girder bending moments are noticeably insensitive to variations of k. As a fair approximation to be judged by results such as those given later in this paper, we will assume that for a uniform suspension cable

$$k = 20\,\frac{w}{d} \qquad (4)$$

We have thus derived a very simple expression for a foundation stiffness approximately equivalent to the gravity stiffness of a cable.*

S4 Treatment of Suspension Bridge by Proposed Analogy

We are now in a position to treat a given bridge problem by utilising existing results for beams on uniform elastic foundations. Taking the idealised case of S2—that is, a single span bridge with cables as though fixed in space at the tower tops and a continuous uniform stiffening girder simply supported at its ends—we have at once from the collection of results in Hetenzi's book[10]

(i) For a single load P at a distance a from one end ($x = 0$) of the bridge

$$M = M_0 - \frac{2PL}{\pi^2}\sum_{n=1}^{n=\infty}\left\{\frac{\sin\frac{n\pi a}{L}\cdot\sin\frac{n\pi x}{L}}{n^2\left(1+n^4\cdot\frac{\pi^4 EI}{kL^4}\right)}\right\} \qquad (5)$$

$$y = \frac{2PL^3}{\pi^4 EI}\sum_{n=1}^{n=\infty}\left\{\frac{\sin\frac{n\pi a}{L}\cdot\sin\frac{n\pi x}{L}}{n^4+\frac{kL^4}{\pi^4 EI}}\right\} \qquad (6)$$

(ii) For a uniform load of intensity q spread over a length c of the span, starting at a distance a from one end ($x = 0$) of the bridge

$$M = M_0 - \frac{2qL^2}{\pi^3}\sum_{\eta=1}^{n=\infty}\left\{\frac{\left(\cos\frac{n\pi a}{L}-\cos\frac{n\pi(a+c)}{L}\right)\sin\frac{n\pi x}{L}}{n^3\left(1+n^4\frac{\pi^4 EI}{kL^4}\right)}\right\} \qquad (7)$$

$$y = \frac{2qL^4}{\pi^5 EI}\sum_{n=1}^{n=\infty}\left\{\frac{\left(\cos\frac{n\pi a}{L}-\cos\frac{n\pi(a+c)}{L}\right)\sin\frac{n\pi x}{L}}{n\left(n^4+\frac{kL^4}{\pi^4 EI}\right)}\right\} \qquad (8)$$

In these expressions M_0 stands for the bending moment on the girder were the foundation (or cable) not present.

The series solutions given above have been chosen in preference to the direct solutions, both because the latter are reasonably short only in case (i) and because it happens that in practice the series converge so that only the first few terms (4 or 5) of the summations are significant. In utilising these expressions we have simply to calculate M_0 and then to evaluate the summations by taking $k = 20\,\frac{w}{d}$.

To compare the bending moment result for a single load P with that of our dimensional theory, we have first to note that

$$\frac{\pi^4 EI}{kL^4} = \frac{\pi^4}{20}\cdot\frac{EId}{wL^4}.$$

We see then, at once, that for a given value of a and x, expression (5) gives M in terms simply of PL and the ratio $\frac{EId}{wL^4}$; the summation term is in fact the function

$$\phi\left(\frac{EId}{wL^4}\right)$$ of (2) in our dimensional theory.

The elastic foundation analogy leading to the above results can be used to provide another useful way of viewing the parameter $\frac{EId}{wL^4}$. When a load is applied to a long beam resting on an elastic foundation, the "spread" of the resulting disturbance along the length of the beam is governed by the relative stiffness of beam and foundation, represented in our case by the above parameter. Now in most modern suspension bridges the spans are such that the stiffening girder, viewed on

*Assuming its ends are fixed; end displacements due to pulls from adjoining spans, tower flexure, or tower compression, may be approximately allowed for by corrections to k or by separate treatment.[14]

its own, is very flexible compared with the loaded cable, and as a result the effects of local loading on the bridge are commonly restricted to a small proportion of the span. The measure of this tendency, in elastic foundation theory, is $\left(\frac{kL^4}{4EI}\right)^{\frac{1}{4}}$, which defines the rate at which a disturbance dies out along the beam away from the load. This term is directly related to our parameter by the proportionality between k and w/d, and for practical suspension bridges of over 1,000 ft. span has values of 2π to 10π. The "long beam" behaviour noted above is commonly regarded[10] as characteristic of all cases with values of $\left(\frac{kL^4}{4EI}\right)^{\frac{1}{4}}$ in excess of π.

It is thus apparent that for many purposes, particularly for loads spread over lengths that are short compared with the span, the stiffening girder of a suspension bridge may be treated as of infinite length on a continuous foundation, and the end conditions be neglected. It is only when the loading is near an end that much error is likely to be introduced by this assumption, and even in such a case the girder could be treated as starting from the end concerned and of infinite length away therefrom—i.e., as a semi-infinite girder.

Simple direct solutions for the doubly-infinite case exist, and properly used will in many cases give a quick approximate result :

(iii) For a single load P on an infinite beam

$$M = \frac{P}{4\lambda} \cdot e^{-\lambda x}(\cos \lambda x - \sin \lambda x) \quad \ldots\ldots (9)$$

$$y = \frac{P\lambda}{2k} \cdot e^{-\lambda x}(\cos \lambda x + \sin \lambda x). \quad \ldots\ldots (10)$$

where $\lambda = \left(\frac{k}{4EI}\right)^{\frac{1}{4}}$ and x is measured from the point of application of P.

(iv) For a uniform load of intensity q spread over a length $(a + b)$ on an infinite beam

$$M = \frac{q}{4\lambda^2}(e^{-\lambda a} \sin \lambda a + e^{-\lambda b} \sin \lambda b) \quad \ldots\ldots (11)$$

$$y = \frac{q}{2k}(2 - e^{-\lambda a} \cos \lambda a - e^{-\lambda b} \cos \lambda b) \quad \ldots\ldots (12)$$

where M and y relate to the point under the load dividing its length into the parts a and b.

It is of interest to notice the significance of results (iv) in relation to the effect of spreading a load along a given proportion of the span. Provided the load is not close to the ends, conditions there will be unimportant and we can use the expression(11) for M to examine the maximum moment. This will clearly occur at the centre of the loaded length $(a = b)$ and is given by

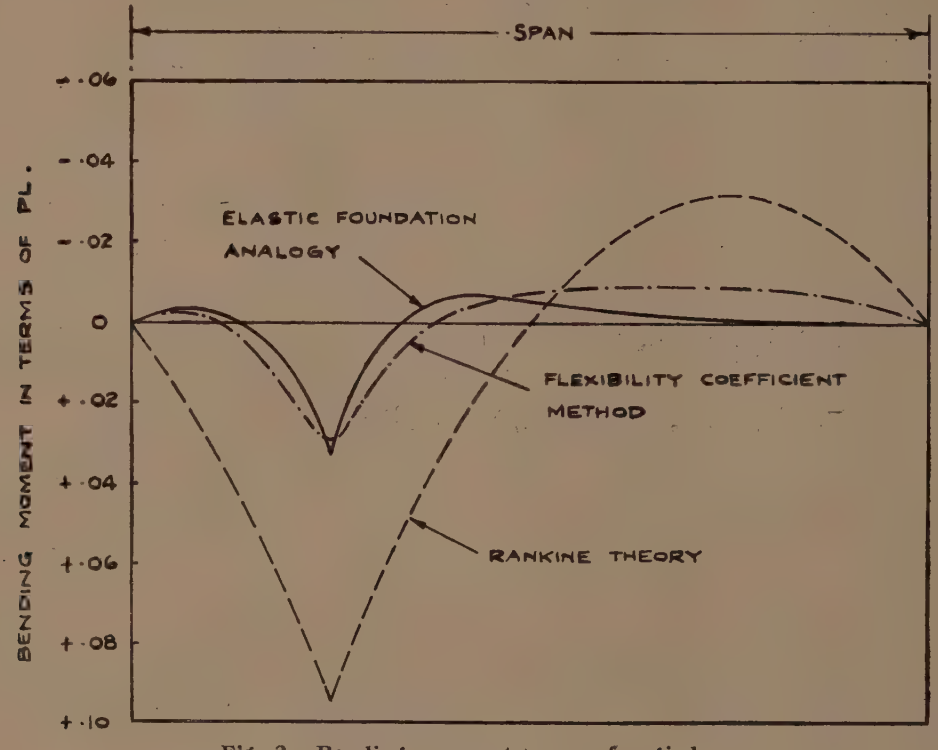

Fig. 2.—Bending moment curves for girder

$$M_o = \frac{q}{2\lambda^2} \cdot e^{-\lambda a} \sin \lambda a,$$

where a is the semi-length of the load. For a given loading intensity M_o will vary with a, attaining a maximum value when

$$\frac{\partial M_o}{\partial a} = \frac{q}{2\lambda} \cdot e^{-\lambda a}(\cos \lambda a - \sin \lambda a) = 0 ;$$

that is, when $\lambda a = \frac{\pi}{4}, \frac{5\pi}{4} \ldots\ldots$ The corresponding significant value of M_o is

$$M_o = 0.161 \frac{q}{\lambda^2} \quad \ldots\ldots\ldots (13)$$

If the Rankine method be applied to the same case —i.e., a load of intensity q over a length $2a$ centrally placed on a span L, the maximum moment at the centre

of the span arises when $a = \frac{L}{4}$ and has a value of

$$M_o = \frac{1}{32} . q . L^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

It will be seen that the relation between (13) and (14) depends solely on λL, i.e., upon the parameter $\left(\frac{kL^4}{4EI}\right)^{\frac{1}{4}}$ and thence upon $\frac{EId}{wL^4}$ of our previous work.

The larger this parameter (corresponding to a low stiffness of the girder compared with the gravity stiffness of the cable), the smaller the maximum value of M_o compared with that given by the Rankine theory. Thus the Rankine theory appears as representing a case where the stiffening girder is very stiff, which is consistent with its inherent assumption of a girder stiff enough to spread a concentrated load over the full cable span, and which explains its decreasing relevance to practice as suspension bridge spans have increased during the last century.

It remains to consider the accuracy obtainable by the use of this elastic foundation analogy. In Fig. 2 bending moment curves are shown for a particular stiffening girder previously analysed[9] by the author by the Rankine method and by the flexibility coefficient method, which may be regarded for the present purpose as giving almost precise results. In the same diagram, for the same conditions, is given the curve obtained by the analogy method, using equation (5) of this paper. The close agreement between this last curve and that deduced by the flexibility coefficient method is apparent. Moreover, the peak bending moment given by equation (9) for the doubly-infinite case is almost coincident with the peak of the flexibility coefficient curve. These comparisons also give a check, though, as has been remarked, not a sensitive one, on the adequacy of equation (4) for k.

S5 Typical Application

Consider a single span suspension bridge with a length of 2,000 ft. and cables having a dip of 200 ft. Assume that one cable and its associated stiffening girder and decking has a total dead weight of 1 ton per ft. run and is called upon to carry a live loading equivalent to $\frac{1}{2}$ ton per ft. run, capable of covering the whole or any part of the span. Let the stiffening girder be steel and of Warren type with a uniform equivalent "moment of inertia" of 5×10^6 in.^{4} (this corresponds to a girder of about 30 ft. depth with booms of about 80 in.^{2} sectional area).

On the basis of the Rankine theory, the maximum moment acting on the girder will arise when the live loading covers the middle 1,000 ft. of the span and has a value, from equation (14), of 62,500 tons/ft.

For the elastic foundation analogy, we may take

$$k = 20\frac{w}{d} = \frac{20 \times 1}{200} = 0.1 \text{ tons per ft.}^2,$$

and so determine

$$\lambda = \left(\frac{k}{4EI}\right)^{\frac{1}{4}} = \frac{1}{369} \text{ ft.}^{-1}$$

For a maximum moment, on the basis of the approximate analysis just given

$$\lambda a = \frac{\pi}{4},$$

whence $a = 290$ ft., indicating that the maximum bending moment is likely to occur when a length of about 580 ft. is covered by the live load. The corresponding maximum moment is, from (13),

$$M_o = 0.161 \frac{q}{\lambda 2} = 11{,}000 \text{ tons/ft.}$$

From this preliminary examination on the elastic foundation basis, it is clear that a shorter length of loading than the Rankine 1,000 ft. is likely to give rise to the maximum bending action on the girder, and that the actual maximum moment will be only a fraction of the Rankine value. It is therefore worth proceeding to a more accurate calculation on the analogy basis, allowing for the end condition effects ignored by our first approximation.

Assuming that the critical length of loading is 580 ft. and placing this centrally on the span, equation (7) of case (ii) in S4 is applicable, for which we have $q = \frac{1}{2}$ ton per ft., $a = 710$ ft., $c = 580$ ft., and

$$\frac{2qL^2}{\pi^3} = 129{,}000 \text{ tons/ft.}$$

The maximum moment will occur at $x = 1{,}000$ ft., where M_o, for the simply supported beam condition, is 124,000 tons/ft. We thus have at the centre

$$M = 124{,}000 - 129{,}000 \sum_{n=1}^{n=\infty} \left\{ \frac{(\cos 1.12n - \cos 2.02n) \sin 1.57n)}{n^3\left(1 + \frac{n^4}{35 \cdot 3}\right)} \right\}$$

The summation here gives

$$\Sigma = .853 + 0 + .022 + 0 + .001 + = .876,$$

whence $M = 124{,}000 - 113{,}000 = 11{,}000$ tons/ft. Thus to the slide rule accuracy of the numerical working, the approximate result is confirmed. Were the loaded length to be increased to 1,000 ft. as suggested by Rankine's analysis, the full analysis just used (equation (7)) gives a maximum bending moment of 7,700 tons/ft. ; this justifies the indication of the approximate analysis that a smaller loaded length* (580 ft.) than Rankine's 1,000 ft. would, for central loading, give the maximum bending action on the girder.

It is of interest to use the same formula (7) of case (ii) to determine the effect of moving a given loaded length from the central region to one end. For practical ratios of girder to cable stiffness, the maximum bending action will remain in the region of the centre of the loaded length. Adopting this as a rough guide, we have for the two loaded lengths considered :

$a = 0$, $c = 580$ ft., M at $x = 300$ is 15,500 tons/ft.

$a = 0$, $c = 1{,}000$ ft., M at $x = 500$ is 12,000 tons/ft

Again the shorter loading* gives the higher figure of 15,500 tons/ft., which itself is higher than the figure of 11,000 tons/ft. when the end conditions and location of the loading are ignored. Bending moment curves for the two cases involving loading along 580 ft. of the span are shown in Fig. 3.

For finding the maximum bending action on the girder, therefore, a simple approximate process appears to be

*These results are consistent with the design loaded lengths suggested by Hardesty and Wessman.[14]

(1) determine the critical length c from the relation

$$\lambda a = \frac{\pi}{4},$$

(2) consider this loaded length as at one end of the girder and find from equation (7) the bending moment at $x = \frac{1}{2}c$.

A full exploration over the range of possible loading cases by equation (7) would, of course, lead to a more accurate result. In our example the approximate process has given a maximum bending moment of 15,500 tons/ft. compared with 62,500 tons/ft. given by the Rankine method.

It remains to determine the influence of the live loading on the tension in the cable. It is clear from the elastic foundation analogy that this will be a maximum when the span is fully covered. In this case, the reactions on the supports at the ends are[10]

$$R = \frac{q}{2\lambda} \cdot \frac{\sinh \lambda L + \sin \lambda L}{\cosh \lambda L + \cos \lambda L} \quad \ldots\ldots (15)$$

method would have taken $q' = q$, whence $H = 1{,}250$ tons.

The loading in the vertical suspension rods will at any point correspond to the loading on the elastic foundation and will be measured by the local deflection y. The magnitude of y is in general given by equation (8) and will tend to be greatest when some central portion of the span only is loaded. In this case the deflection equation (12) will give a good approximation. For a maximum value for the given length $(a + b)$, $a = b$ and

$$y = \frac{q}{k}(1 - e^{-\lambda a} \cos \lambda a),$$

where a is the semi-length of the load. This is a maximum when $\lambda a = \frac{\pi}{4}$ as in the bending moment case, and then

$$y = y_0 = .678 \frac{q}{k} \quad \ldots\ldots (18)$$

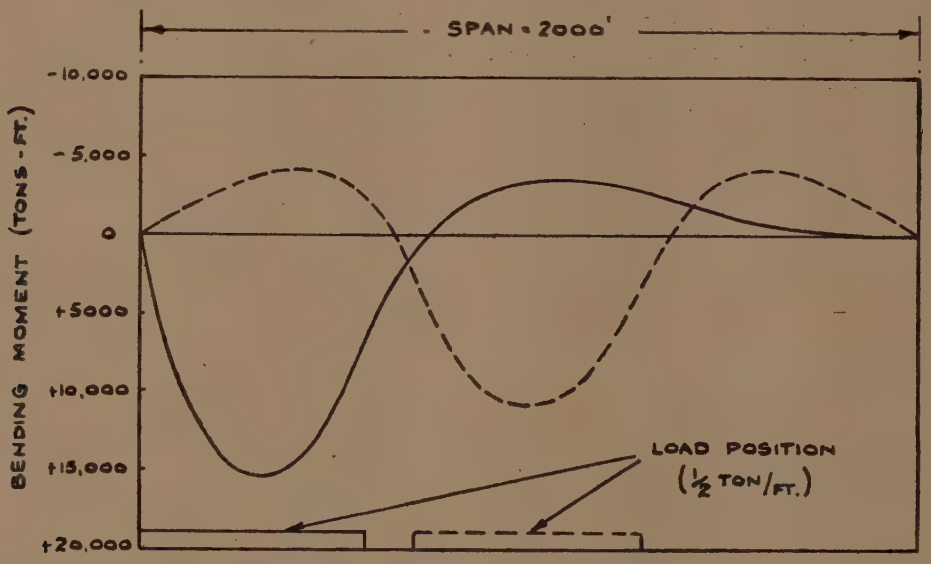

Fig. 3.—Bending moment curves, for example of S5

which for our example gives $R = 91$ tons. Thus of the full 1,000 tons on the girder, 182 tons are carried by its end supports and the remaining 816 tons by the elastic foundation—i.e., by the cable. The distribution of this loading across the span will not be uniform, and is given by the expression for the deflection[10] for this case

$$y = \frac{q}{k}\left\{1 - \frac{\cosh\lambda x \cos \lambda x' + \cosh \lambda x' \cos \lambda x}{\cosh \lambda L + \cos \lambda L}\right\} \quad (16)$$

where $x' = L - x$. This is plotted for our example in Fig. 4 and is, in fact, not greatly different from uniform. Thus the maximum increase in the horizontal component of the cable tension will be given approximately by

$$H = \frac{q'L^2}{8d}, \quad \ldots\ldots (17)$$

where $q' = \frac{816}{2000} = .408$ tons per ft. This gives $H = 1{,}020$ tons as the maximum live load addition to the tension of 2,500 tons due to the dead load. The Rankine

This very convenient general expression gives a maximum loading on the rods of $ky_0 = .678\ q$ tons per ft. of girder ; thus in our case, the maximum loading on the rods, if spaced at 40 ft. intervals, would be $.678 \times \frac{1}{2} \times 40 = 13.5$ tons each.

Were the Rankine method to be applied to this problem, we might consider a central length of the span covered and note that, if q'' is the uniform upward force per ft. run supplied by the rods over the whole span

$$q'' L = qc,$$

whence it is clear that q'' will be greatest when the whole span is covered $(c = L)$. In that case $q'' = q$, and for 40 ft. spacing of the rods the load would be $\frac{1}{2} \times 40 = 20$ tons each.

It will be noted in general that whereas the Rankine method gravely overestimates the girder bending moments, it does not so seriously overestimate the cable and suspension rod tensions.

S6 Flexural Vibrations

By way of a tailpiece to the foregoing development and application of the elastic foundation analogy, it is

f interest to consider its relevance to the problem of redicting the natural frequencies of a suspension bridge ructure. The difficulties in this problem arise from ne fact that no precise explicit expression is known for he natural frequencies of a suspension cable[12], and that hese frequencies correspond to modes that are not aturally linked with those of a simple beam.

The flexural frequencies of a beam with hinged ends nd resting throughout its length on an elastic foundation f modulus k are given[11] by

$$p_i^2 = \frac{a^2\pi^4}{L^4}(i^4 + \beta), \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

where p_i = circular frequency of the ith mode, the shape

of which is defined by $y_i = A_i \sin \frac{i\pi x}{L}$,

and $a^2 = \frac{EIg}{w}$,

$$\beta = \frac{kL^4}{\pi^4 EI}.$$

$$n' = \frac{\sqrt{\alpha}}{2\pi}\sqrt{\frac{g}{d}}\sqrt{1 + \frac{2\pi^4}{3\alpha}R} \quad . \quad . \quad . \quad . \quad (21)$$

$$n'' = \frac{\sqrt{\alpha}}{2\pi}\sqrt{\frac{g}{d}}\sqrt{1 + \frac{27\pi^4}{8\alpha}R} \quad . \quad . \quad . \quad . \quad (22)$$

$$n''' = \frac{\sqrt{\alpha}}{2\pi}\sqrt{\frac{g}{d}}\sqrt{1 + \frac{32\pi^4}{3\alpha}R} \quad . \quad . \quad . \quad . \quad (23)$$

These results may be compared with the expressions deduced by the author[9] from Steinman's approximate formulæ[13]

$$n' = 0.35\sqrt{\frac{g}{d}}\sqrt{1 + 13.2R} \quad . \quad . \quad . \quad . \quad (24)$$

$$n'' = 0.49\sqrt{\frac{g}{d}}\sqrt{1 + 29.6R} \quad . \quad . \quad . \quad . \quad (25)$$

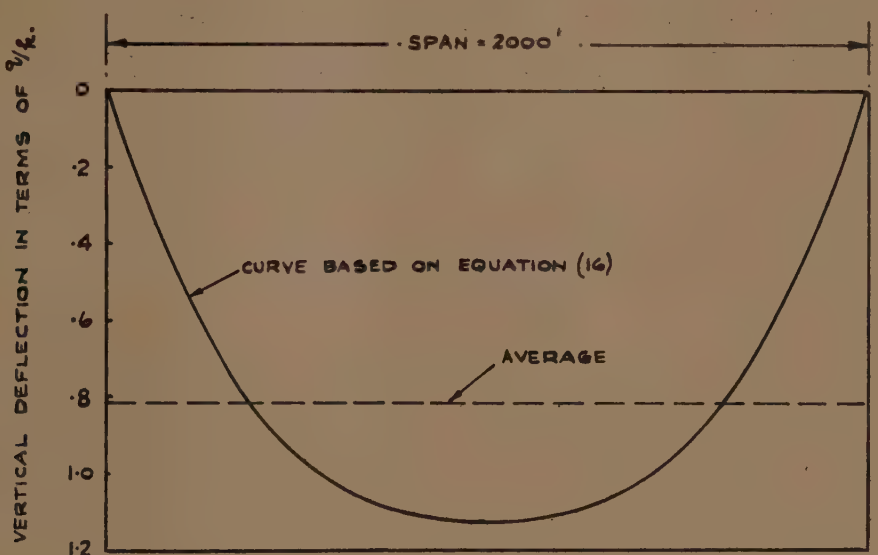

Fig. 4.—Distribution of vertical deflection, for example of S5

Rewriting this expression for p_i in terms of the natural frequency n_i, we have

$$n_i = \frac{1}{2\pi}\sqrt{\frac{gk}{w}}\sqrt{1 + i^4\frac{\pi^4 EI}{kL^4}}.$$

Inserting $k = \alpha \cdot \frac{w}{d}$ (where α was chosen as 20 for our static applications of the analogy) and $R = \frac{24EId}{wL^4}$, this becomes

$$n_i = \frac{\sqrt{\alpha}}{2\pi}\sqrt{\frac{g}{d}}\sqrt{1 + \frac{\pi^4}{\alpha} \cdot i^4 \cdot R}, \quad . \quad . \quad . \quad (20)$$

whence (noting that $i = 1$ is inadmissible as it corresponds to a mode depending essentially upon extensions of the cable) for the first three natural modes we have

$$n''' = 0.71\sqrt{\frac{g}{d}}\sqrt{1 + 52.7R} \quad . \quad . \quad . \quad (26)$$

It will be seen at once that our analogy has given the same form of expression as the other approximate analysis, and almost precisely the same result for the particular frequency n''', for with $\alpha = 20$, equation (23) gives

$$n''' = 0.71\sqrt{\frac{g}{d}}\sqrt{1 + 52.0R} \quad . \quad . \quad . \quad (27)$$

but a constant value of α is incapable of giving an equal degree of accuracy for all modes. This is partly because of the empirical nature of the choice of a value for α, but more fundamentally because of a peculiar property of the "elastic foundation" itself. This idealised foundation, as a matter of definition, is such that a deformation in one region has no influence upon that in another (due to the absence of shear stiffness in the idealised foundation, which can be represented by a series of separate vertical springs), and as a result the

energy stored in the foundation by a given sinusoidal deformation varies at the amplitude of the deformation but is independent of its wave length. It is this peculiar property that renders the constant $\frac{\sqrt{\alpha}}{2\pi}$ in equations (21), (22) and (23) invariant with i and so limits the generality of the analogy in this application.

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Vibrations in Foundations

THE FACTORS WHICH AFFECT THE NATURAL FREQUENCY OF FOUNDATIONS, AND THE EFFECT OF VIBRATIONS ON THE BEARING POWER AND SETTLEMENT OF FOUNDATIONS ON SAND

By W. Eastwood, B.Eng., Ph.D., A.M.I.Struct.E.

Synopsis

The paper discusses previous work concerned with vibrations in foundations and then describes two investigations by the author concerned with model foundations on sand.

The first of these was intended to provide information concerning the various factors which affect the natural frequency of vertical vibration. Besides testing footings of different shapes and sizes subject to various intensities of dead loading, the effect of inundating the foundation has been studied and also the effect of a layer of more rigid strata at different depths beneath the footing.

In the second investigation the foundations were subjected to controlled forced vibrations. The effect of these vibrations on the settlement and ultimate load of footings of different shapes and sizes has been measured. Some of the tests were repeated with the footings inundated.

Historical

Vibrations in foundations may be due to either one of two causes. They may be caused by a train of waves passing through the ground, or they may be set up by fluctuating loading on the foundation itself.

Trains of waves passing through the ground may be generated by some natural phenomenon such as an earthquake, or may be man-made. Typical instances of the latter are ground vibrations due to blasting, piling, or detonation of bombs, all of which have been investigated in recent years.

Ground vibrations due to blasting have been investigated by the U.S. Bureau of Mines[1]. Broadly speaking, the conclusions regarding the effects of vibrations on structures were that minor damage such as plaster cracking could be expected if the maximum acceleration during a cycle of vibration exceeded $\frac{1}{10}g$, whilst major structural damage might be expected if the acceleration was in excess of g. But in a more recent paper, Crandell[2] maintains that acceleration is not the only criterion of the severity of vibration. He maintains that a quantity which he calls the "Energy Ratio" is more critical. The Energy Ratio is defined as

$$\frac{(\text{Acceleration})^2}{(\text{Cycles per second})^2},$$

and he states that if the ratio is less than 3 (ft. sec. units) there should be no damage, but if the value is in the region of 10 there may be considerable damage, particularly to brickwork. Crandell's criterion certainly appears to be more correct than that of the Bureau of Mines at low frequencies. He quotes the instance of an earthquake in which the maximum acceleration was 3 ft. per sec.[2] with a frequency of 1 cycle per sec. The acceleration was less than the Bureau of Mines lower limit of $\frac{1}{10}g$, and yet considerable damage was caused to structures, particularly brick ones. The Energy Ratio of 9 agrees with Crandell's value of 10 for considerable damage.

In connection with blasting, Crandell showed that the Energy Ratio of a train of vibrations caused by blasting could be obtained in advance by use of the formula

$$\text{Energy Ratio} = \left(\frac{50}{D}\right)^2 C^2 K$$

where D is the distance from the charge C lb. of explosives. The constant K would have to be obtained in advance for any particular locality by measurement.

Trains of waves due to blasting have been used for some time, of course, in ground exploration. Shepard and Haines[3] have reported a survey of the different strata, together with an estimate of their properties, in the region of the St. Laurence River Project, seismic type recorders being used to pick up the waves at different distances from an explosive charge.

ibrations in foundations due to a train of waves ving horizontally through the ground are not, vever, the main concern of this paper. The experi- nts described later were all concerned with vertical rations of the foundation, generated either by a single w resulting in free oscillations, or by a continuously plied sinusoidal force giving forced oscillations. prations generated in one of these two ways are the s most commonly found in practice.

n studying the effects of vibrations on a foundation re are really two separate problems. The first of se is to determine the factors which effect the natural quency of oscillation, since, unless the vibratory force applied at a frequency sufficiently near the natural e to give resonance, the oscillations are unlikely to come very exuberant. The second problem is to imate the effect on the settlement and ultimate load the foundations if the vibrations are built up by sonance.

The factors which affect the natural frequency may clude the shape of the foundation, its dimensions, the d which it is carrying, and, of course, the soil proper- s. Some attempt has already been made to examine e effects of these factors by various investigators.

In the period 1928 to 1939, Degebo (Deutsche Gesell- haft fur Bodenmechanik) experimented with an ctrically driven oscillator which was mounted on her of two baseplates. The baseplate most frequently ed was 1 sq. metre in area and the combined weight the plate plus the vibrator 1540 kilo. The other base- ate was 0.25 sq. metres in area and gave a total weight 1356 kilo. The vibrating mechanism could be driven any speed up to 1875 cycles per minute with a maxi- um out of balance force of 1200 kilo. Tests were made all kinds of soil, peat, clay, sand, gravel, and also on ndstone, but unfortunately no attempt appears to ve been made to measure such important factors as e density and moisture content of the soil. Nor does y attempt appear to have been made to measure e thickness of the soil layer and to identify the under- ng material. It is probable that considerable varia- n in the results would occur because of the variation these factors from one site to another, and even from y to day on the same site.

Some of the conclusions of Degebo now seem to be justified in the light of later knowledge. For example, e following quotation is taken from the British nistry of Works translation of a paper by Hertwig, uh and Lorenz[4] describing the variation of resonant quency with the dimensions of the foundation : . . . It can be stated with certainty that the value the resonant frequency) increases with the increase the surface at a constant centrifugal force. This rease is understandable, since the increase of the face of the base means supporting the mass point by a onger, stiffer foundation. If the foundation is nsidered to be a spring, an increase, of the base responds to a stiffening of the spring. . ." It is ll-known nowadays that in a static plate bearing test e stiffness of the soil measured as a load per unit area unit of deflection, decreases as the plate diameter reases. Exactly the same happens under dynamic flections, as will be seen later, so that a higher quency should have been obtained with Degebo's small te than with the big one. The probable explanation their result is that the small plate had an applied ad load oscillating with it almost four times as great unit area as that on the large plate, thus very much lucing the natural frequency.

Despite the above criticism of the design of Degebo's periments and some of the conclusions arrived at, the author appreciates the great value of its work in establishing that there was such a thing as a natural frequency, and for realising that the whole problem could be tackled in a scientific way.

Whilst the Degebo tests were being carried out the Institute for Engineering Foundation Research ('Vios ') were doing similar tests in Russia[5]. The foundations used were quite large, having base areas of 2, 4 and 8 square metres respectively. The main object in analysing the results appears to have been to see if the modulus of subgrade reaction could be obtained dynamically. When making the calculations it seems to have been assumed that the mass of the soil vibrating with the foundation was small compared with the mass of the foundation itself and it was therefore neglected. Indirectly, however, it was concluded that the mass of the soil could not be neglected and that its value must lie between two-thirds and one and a half times that of the foundation. This was very much less than the value estimated by Degebo who had reported that it was between four and ten times that of the vibrator, depending, so they said, on the vibratory force.

This discrepancy between the two values for the effective mass of the earth is an important one since the natural frequency is very much dependent on it. Assuming that the foundation ground system is analogous to the spring-mass system shown in Fig. 1a, the natural frequency of oscillation is given by $p/2\pi$ where

$$p^2 = \frac{F}{M_f}$$

F being the stiffness of the spring, and M_f the vibrating mass. If, however, the spring is assumed to have a

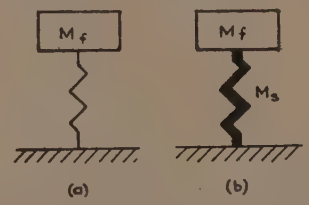


Fig. 1.—Mass supported by springs of (a) negligible weight, and (b) considerable weight

mass as in Fig. 1b, then the natural frequency is reduced. This reduced frequency can be calculated by exact analysis but a fairly close approximation is obtained by assuming that the spring has an effective mass M_s concentrated with M_f. (Note that the effective mass M_s is not the actual mass of the spring.) The frequency is now given by

$$p^2 = \frac{F}{M_f + M_s}$$

Now it will be seen that if M_s is large compared with M_f in the foundation ground system changes in M_f will have little effect on the frequency. The relative value of M_f and M_s is obviously one of the things on which more information is needed, therefore. As will be shown later, the difference between the magnitudes of M_s reported by Vios and Degebo was probably due to the fact that F is not constant in the case of a foundation. Indeed, using the same method of obtaining M_s as was used by Degebo and Vios has given a negative value of M_s in certain of the author's tests.

Andrews and Crockett[6], and Crockett and Hammond[7] have also made measurements of natural frequencies, using a vibrograph to pick up the oscillations in the vicinity of large hammers. These frequencies are roughly the same order as those reported by Degebo

for any particular type of soil. Crockett and Hammond also state that for any particular type of ground they got the same natural frequency irrespective of the size of the foundation, the largest tested having an area of 2,500 sq. ft. It is difficult to put a true interpretation on this statement, however. For various reasons it does not seem logical that all foundations whatever their size and weight should have the same natural frequency. For example it would be necessary that the spring stiffness of the ground in lb. per sq. in. per in. of deflection would have to be constant for all foundation widths. That this can be so is very unlikely, since the soil must behave at least partially like an elastic mass to set up oscillations, and as is well-known, a foundation on an elastic mass stresses soil to a depth proportional to the foundation width. This would cause the spring stiffness per unit area to decrease as the foundation size increased. In addition, if a very narrow foundation is considered, it would be necessary for the effective mass of soil vibrating with it to be very large if the natural frequency were to remain constant whether the footing were loaded with 3 ton per sq. ft. or were virtually unloaded. This would mean that the soil must be highly stressed to a depth many times the width of the foundation which is contrary to common knowledge. It seems, therefore, that in Crockett and Hammond's tests the relation between the weight of the foundation and its width had quite fortuitously varied from one foundation to another in such a way that they gave roughlythe same frequency. It should be pointed out in fairness to Crockett and Hammond that they were merely reporting what they had observed, and were not claiming that it was of necessity always true. It is, however, necessary that its universal truth should be tested as soon as possible, and this has been done on the model scale in the tests described later.

One interesting fact which was brought to light by Crockett and Hammond is that resonance can occur at harmonics or even sub-harmonics. They report, however, that resonance could not be obtained at all the frequencies peculiar to one site on any particular day.

It appeared that the presence of these harmonics was probably dependent on changes in the moisture content and hence of the elastic properties of the ground. Even when present the harmonics were not nearly so pronounced as the fundamental.

There has been little attempt to develop an analysis which would enable the natural frequency of vibration to be calculated in advance. One difficulty in developing such a theory is the one common to many soil mechanics problems, that the soil cannot be assumed to behave perfectly elastically. But Degebo, Vios, and Crockett and Hammond have all agreed that the mass of soil which vibrates with the foundation must bear some relation to the bulb of stress (see Fig. 2) which gives the stress distribution under a uniformly loaded area on an elastic medium. None of them have stated what that relation is. Indeed, as mentioned earlier, estimates of the mass of the vibrating soil relative to the mass of the foundation given by Degebo differ greatly from those given by Vios.

As far as the spring stiffness of the ground is concerned, none of the previous investigators has suggested a method of calculating it from first principles, although it has been inferred that it too must depend on the bulb of stress.

Two analytical methods of approach have been suggested, however, which are of interest. Tschebotarioff and Ward[8] have suggested that there is a logarithmic relation between the area of any foundation and what they call the "reduced natural frequency." The standard formula relating the frequency of a spring born mass to the spring stiffness was quoted earlier in the form

$$p^2 = \frac{F}{M}$$

where the frequency is $p/2\pi = N$ say.

The formula can also be written

$$N = \frac{1}{2\pi}\sqrt{\frac{F}{M}}$$

and if M is replaced by $\frac{W_f}{g}$, where W_f is the weigh of the foundation, plus $\frac{W_s}{g}$ the equivalent mass the vibrating soil we get

$$N = \frac{1}{2\pi} \cdot \sqrt{\frac{Fg}{W_f + W_s}}$$

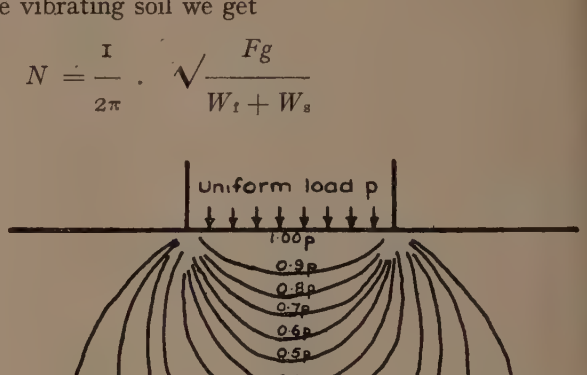


Fig. 2.—Vertical stresses under circular loaded area on an elastic media

The stiffness F of the ground can be replaced by kA where k is the stiffness per unit area and A is the area the formula then becoming

$$N = \frac{1}{2\pi} \cdot \sqrt{\frac{kA_g}{W_f + W_s}} \quad \ldots \quad \text{(i)}$$

$$\text{i.e., } N = \frac{1}{2\pi}\sqrt{\frac{kg}{1 + \frac{W_s}{W_f}}} \cdot \sqrt{\frac{A}{W_f}} \quad \ldots \quad \text{(ii)}$$

Now $\frac{A}{W_f}$ is equal to w, the load on the foundation divided by its area. Thus at one ton per sq. ft. the frequency would be

$$N_r = \frac{1}{2\pi} \cdot \sqrt{\frac{k_g}{1 + \frac{W_s}{W_f}}} \cdot \sqrt{1} \quad \ldots \quad \text{(iii)}$$

where N_r is what Tschebotarioff calls the "reduced natural frequency." The value of N_r can be obtained

r any particular foundation the natural frequency of ich is known from the equation

$$N_r = \sqrt{w} \cdot N$$

Tschebotarioff worked out values of N_r from this uation using the published results of Vios, Degebo and hers, and also the results of some tests which he mself carried out in a Californian bearing ratio test ould (these tests are described later) and plotted them ainst the area of the foundation on logarithmic paper. e claims that the results lie nearly enough on a straight ne, but actually the points are anything up to 100 per nt. high or 50 per cent. low, the errors being masked the logarithmic scale. This would be expected, owever, when it is considered that some of the points fer to piled foundations, and others to tests in a very nall mould.

There are other possible criticisms of this piece of ork in the opinion of the author.

First, the reduced natural frequency is much more kely to depend on the width of the foundation than n its area. For example, the author's tests described ter show that for the same applied load per unit area ne natural frequency of a 12 in. by 3 in. model footing the same as that for a 24 in. by 3 in. model footing. hus, they will also have the same reduced natural requency even though the area of one is twice that of ne other. It is suggested, therefore, that in Tschebotarioff's graph the reduced natural frequency should e plotted against the least dimension of the footing.

Further, referring to equation (iii), it will be seen that n order to obtain the same value for N_r for a given oundation, whatever the applied load, the expression

$$\sqrt{\frac{k_g}{1 + \frac{W_s}{W_f}}}$$

ust be constant. This means that either W_s must ncrease at exactly the same rate as W_f or alternatively hat W_s is always negligible compared with W_f. The tter is impossible (consider for example the case of an

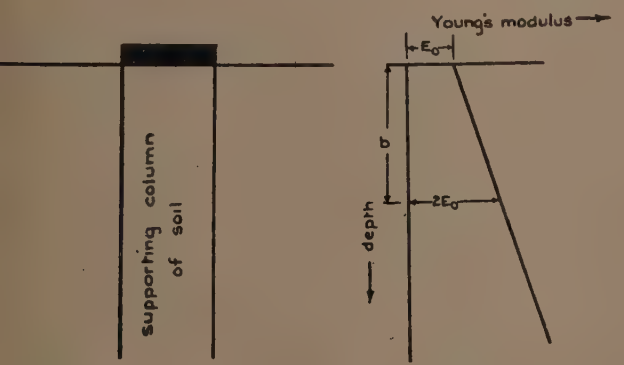


Fig. 3.—Professor Christopherson's assumed variation with depth of Young's modulus of supporting column of soil

xtremely lightly loaded foundation consisting of othing more than a slab) and the former extremely nlikely.

It seems to the author that computation of further N_r alues as more data becomes available is unlikely to how a definite relation between N_r and either the oundation area or the foundation width.

A more promising analytical approach has been uggested by Prof. Christopherson in the discussion on he paper by Crockett and Hammond. He makes a asic assumption that Young's modulus for the ground increases linearly with depth, the value being E_0 at the surface, $2E_0$ at depth b, $3E_0$ at depth $2b$ and so on. The foundation is assumed to be supported by a column of earth as shown in Fig. 3. In actual fact the soil on either side of the column will vibrate, but he assumes that this soil will increase both the vibrating mass and the stiffness of the spring so that its total effect on the frequencies may be negligible. He then shows that the soil can vibrate internally with any one of a number of

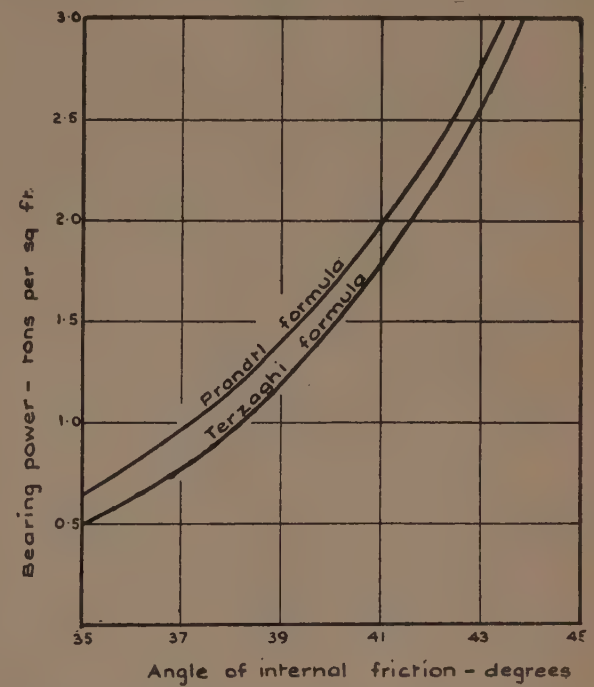


Fig. 4.—Variation of bearing power with angle of internal friction for 6-in. wide footing on sand according to Terzaghi and Prandtl

frequencies (damping being ignored). For a foundation which can be assumed to have negligible weight they fall into two series as follows :—

$$n = \frac{0.175}{b}\sqrt{\frac{E_0}{\rho}}\ ;\ \frac{0.432}{b}\sqrt{\frac{E_0}{\rho}}\ ;\ \frac{0.604}{b}\sqrt{\frac{E_0}{\rho}}\ \text{etc.}$$

and

$$n = \frac{0.305}{b}\sqrt{\frac{E_0}{\rho}}\ \ \frac{0.558}{b}\sqrt{\frac{E_0}{\rho}}\ ;\ \frac{0.810}{b}\sqrt{\frac{E_0}{\rho}}\ \text{etc.}$$

The possibility of there being several natural frequencies agrees with the findings of Crockett and Hammond. It is probable that rainfall or changes of temperature caused a variation with depth in Young's modulus for the sites concerned, thus conforming with Professor Christopherson's basic assumption.

When a foundation weight is applied to the surface, Professor Christopherson's analysis still gives a series of frequencies, but the numerical values depend on the foundation weight, and have to be calculated from tabulated values of Bessel functions.

Unfortunately, Professor Christopherson's theory gives frequencies which are independent of the foundation size. As will be seen from the experimental results given later in the paper, this is not in accordance with observed phenomena, at least for model foundations.

His fundamental assumption that Young's modulus increases uniformly with depth is also unlikely to hold except in the case of a thick bed of consolidated clay.

Turning now to the problem of the effects of vibration on the settlement and ultimate load of foundations, it appears that very little research has been done. One investigation concerned with the Californian bearing ratio test, carried out by Tschebotarioff and McAlpin[9] is of interest. When applying pavement design methods to hardstandings for aircraft it is quite usual to increase the design thickness by some arbitrary figure between 20 and 50 per cent. This is necessary because it is found that the vibratory loading caused by stationary aircraft when warming up is more likely to cause failure

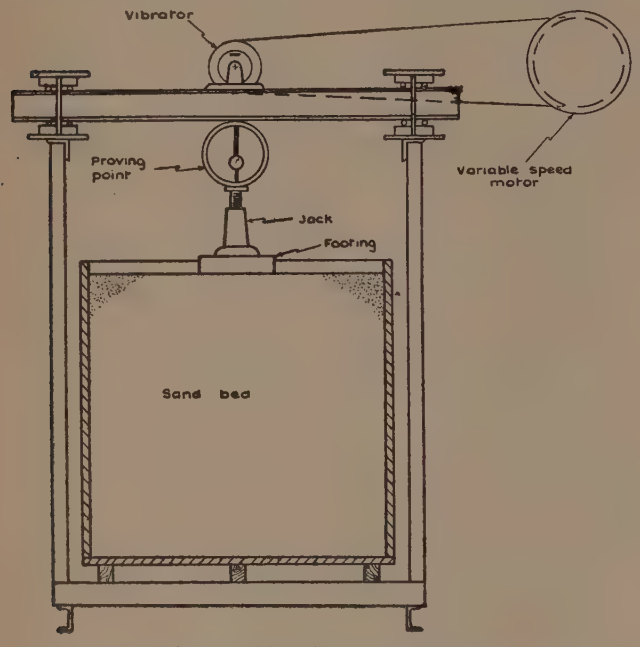


Fig. 5.—Sand box and loading frame showing arrangement for forced vibration tests
[Erratum: For "proving point" above read "proving ring."]

of the pavement than the wheel loads of the same aircraft when landing and taking off. The actual magnitude of the vibratory forces is not large. R. K. Bernhard measured these forces for Tschebotarioff and found that for most aircraft they were generally between 1 and 2 per cent. of the static wheel load. Occasionally they might reach 4 or 5 per cent. if resonant vibrations were occurring in the whole aircraft frame, this condition usually being obtained at engine speeds rather lower than the normal warm-up speed. In addition, the mean wheel load is reduced by about 15 per cent. during warm-up by the action of the slip stream on the underside of the wings. It appears, therefore, that 85 per cent. of the normal static load with a superimposed vibratory load of plus or minus 1-2 per cent. has more effect on the pavement stability than the full static load alone.

The purpose of Tschebotarioff's experiments was to see whether the various pavement design methods could be modified to include the effects of vibratory loads. He built a special version of the C.B.R. apparatus in which the plunger was subjected to dead loading plus the fluctuating load generated by a vibrator. As the plunger was acting on soil in a relatively small mould, the results of his tests will give no direct indication of the behaviour of an actual foundation, but they are useful in indicating the manner in which vibration can increase settlement.

On a well-graded sand it was found that reducing the dead load by 15 per cent. and then superposing a vibratory load of $\pm$ 2 per cent. gave approximately 15 per cent. higher penetrations of the plunger. In a repeat test on a poorly graded sand of low density the penetration was increased by 140 per cent. Varying the frequency of vibration had little effect provided the total number of repetitions was the same. On a clay soil, however, the plunger penetration was sensibly constant whether vibratory loading was used or not.

Some tests were also carried out in which the superposed vibratory force was said to be $\pm$ 25 per cent. of the static load. As would be expected, the penetration of the plunger into the sand was very much increased. It appears, however, that the dead load on the plunger had quite a considerable amplitude of vibration more or less in phase with the oscillating force from the vibrator. Measurements were not taken of this amplitude of vibration, but if it were only say 0.001 in. at 3000 cycles per minute the inertia force of the dead load would amount to $\pm$ 25 per cent. of its own weight. If this were in phase with the force from the vibrator the variation would therefore be $\pm$ 50 per cent., not $\pm$25 per cent. as stated by Tschebotarioff.

Despite the doubts about their quantitative value, and the fact that his tests were not concerned with an actual foundation, Tschebotarioff's results do indicate that vibrations can be tremendously important in the case of foundations on sand, but are not so important in the case of clays.

Evidence of the settlement caused by vibratory loading on sands has also been provided by compaction tests made at the Road Research Laboratory of the D.S.I.R. Certain types of vibrating rollers and vibrating plates have been shown to give good compaction of sandy soils. Again, however, the magnitude of the vibratory force is not known, since the amplitude of the roller or plate and hence its inertia force was not measured (as far as is known to the author).

Regarding the ultimate load of the foundation, it might be expected that for footings on sand the value would be very much reduced by vibration. It is well-known that slight vibration of a shear-box apparatus gives reduced values for the angle of internal friction and, according to the usually accepted theories, a change of 1° may cause a 20 per cent. reduction in the ultimate load of a footing (see Fig. 4). No actual measurements

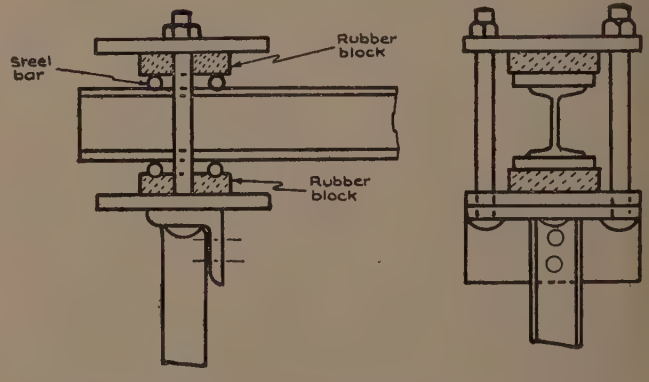


Fig. 6.—Detail of rubber mounting for loading beam

of the reduction in bearing power under controlled vibrations appear to have been made by previous investigators.

Aims of Present Investigation

From the foregoing it will be seen that information is needed on many points concerning both the factors which affect the natural frequency, and also the effect of vibration on the settlement and ultimate load of foundations.

Concerning the natural frequency some of the more portant unknowns are

(i) What is the effective mass of earth which may be said to vibrate with the foundation ?

(ii) How is the " spring stiffness " of the ground affected by the size of the foundation ?

(iii) How is the spring stiffness affected by the shape of the foundation ?

(iv) How are the answers to the above questions affected by inundation ?

(v) What is the effect of a rigid layer of rock at different depths on the natural frequency ?

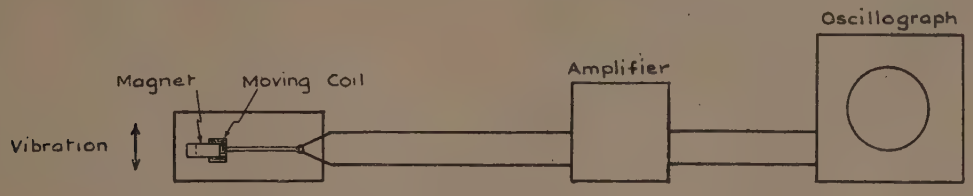


Fig. 7.—Diagram illustrating seismic type pick-up and circuit

Regarding the settlement and the reduction of ultimate ad under the influence of vibration, information is quired on the following points, amongst others.

(i) For a vibration of a given amplitude, is there any particular frequency at which settlement is accelerated and/or the ultimate load markedly reduced ?

(ii) How does settlement increase with time for a given vibration, the footing load being kept constant at some fraction of the static ultimate load ?

(iii) Does the number of repetitions of the vibration affect the ultimate load ?

(iv) How are the answers to the above questions affected if the footing is inundated ?

The object of the experiments hereafter described as to give some indication, at least, what the answer each of the above questions might be with particular ference to foundations on sand. A sandy soil was osen, not because foundation experiments are easier carry out on sand than on clay, but because it is nerally accepted that vibrations have more effect on ndy soils than on clays.

The author wishes to emphasise that, in general, periments on model foundations can be no more than dicative in a qualitive sense of what happens in full- ale foundations. Since all his tests have been on odels the results cannot be interpreted properly until ey have been supplemented by some full-scale tests. evertheless, the fact that the present investigation has en confined to models needs no defence, since the rrying out of a similar programme of tests on a full ale would be very expensive.

Apparatus Used in the Investigation

All the model tests were made on a box of sand in the boratory.

It was obviously essential that the sides of the box ust not interfere in any way with the modes of vibra- on of the footings. There was no means of predicting advance what the ratio of the size of the box to that the footing should be to satisfy this condition. Accord- gly, a box approximately 4 ft. × 4 ft. × 4 ft. was cided upon, the maximum permissible size of the otings to be found as a result of tests. Various sizes model footing were set in oscillation and the waves diating outwards were picked up at various distances om the footing using a seismic type vibration pick-up (described later). It was found that for footings up to 10 in. square, 10 in. diameter, and up to 24 in. × 6 in. in the case of rectangles, the box sides seemed to have negligible effect. The reflection from the box sides was extremely small and could only be detected an inch or two from the box edge. According to the usual mathematical theories, reflection should have been quite high for waves travelling from sand to wood. The low reflection must have been due to the 0.25 in. thick bituminous sheet lining, which was inserted to keep the box watertight, absorbing the energy. A check was also made that when using circular foundations the decay of the waves was the same whether it was measured in a direct line to the side of the box or along a diagonal.

It was apparent that, provided the experiment was concerned solely with the measurement of natural frequencies, the footing could be any size up to those stipulated in the paragraph above. If, however, the test involved loading the footing in order to measure either settlement or the ultimate load the size of the footing must be such that any permanent deformation of the sand would not be interfered with by the sides of the box. From a previous investigation[10], it was known that this would be so if the footings were kept below 24 in. × 4 in.

The rectangular footings were made from lengths of R.S.C. of appropriate width, whilst the squares and circles were cut from half-inch thick steel plate.

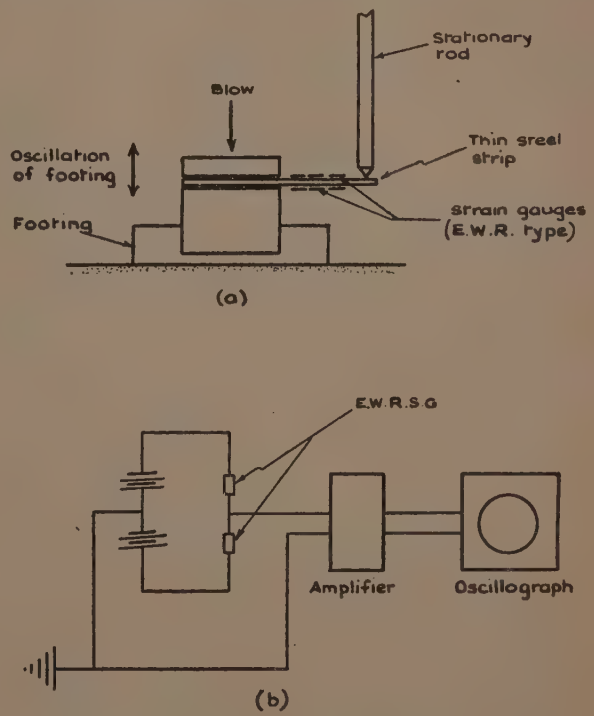


Fig. 8.—(a) and (b). Arrangement of vibration pick-up and circuit diagram

For the tests in which vibrations of controlled amplitude and frequency were superposed on known dead loads, the footings were jacked against a loading frame the cross beam of which was spring mounted (see Figs. 5 and 6). The spring mounting consisted of hard pads of rubber which gave a resistance to vertical movement of about 8 tons per inch. Vertical oscillations of this beam were generated by an out of balance rotating disc mounted as shown in the figure and belt-driven from a variable speed motor. The amplitude of the oscillations

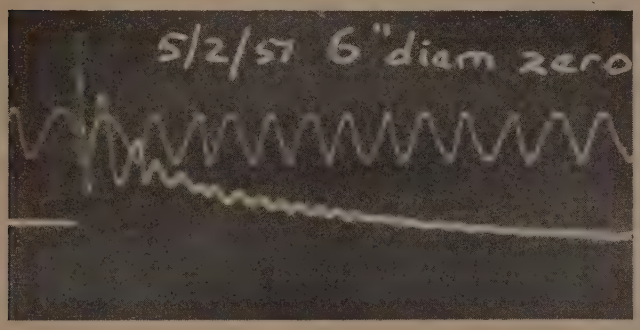


Fig. 9.—Vibration record for 6-in. diameter footing with 50 cycles per second time base

was altered by adjusting the magnitude of the eccentric mass.

The oscillations of the footing and the vibrations in the soil were picked up by means of a De Havilland seismic type pick-up. The impulses from this pick-up were amplified and then fed into a cathode ray oscillograph (see Fig. 7).

For the tests in which the natural frequency of oscillation was to be recorded the foundations were subjected to dead loads instead of being jacked against the loading frame. The oscillations were generated by striking the foundation a single sharp blow with a hammer and were picked up by the arrangement shown in Fig. 8a. Electrical resistance strain gauges were fixed on each side of a steel strip cantilever the encastré end of which moved up and down with the foundation, the free end resting against a stationary rod. The variations in potential across the gauges (see circuit in Fig. 8b) were amplified and fed into a cathode ray oscillograph. The trace on the tube was photographed together with a 50 cycles per second time signal from which the frequency was deduced. A typical record of the vibrations of a footing is shown in Fig. 9.

A grading curve of the sand used in the tests is shown in Fig. 10. Very great care was taken in compacting the sand to ensure that the density was as near constant as possible throughout the box. It was placed dry in 6 in. layers, each layer being compacted to refusal by a Kango electric hammer. Each batch of sand was carefully weighed so that after screeding off at 10 in. below the top edge of the box the average density up to that level was accurately known. Because the electric hammer left a very disturbed surface the compaction of the top 10 in. of sand was carried out by hand ramming with a rammer weighing 18 lb. and of 60 square in. base area. This compaction was continued until the sand in the top 10 in. was at exactly the same density as the sand below that level.

After the sand had been disturbed in each test the top 12 in. was dug up and recompacted to exactly the same average density, which was 108.1 lb. per cu. ft. The specific gravity of the soil particles being 2.65 the voids ratio was 0.525 representing a fairly dense condition.

In the second series of tests with the sand inundated it was obviously desirable to have the sand at the same "dry density" as in the tests on dry sand. But to place it under water to that density over the whole four feet of its depth would have been very difficult, so it was decided to leave the sand ready compacted in the box and to add the water to it. The chief danger in this procedure was that very tiny bubbles of air might be trapped in the interstices between the soil particles. To reduce the possibility of this happening the water was injected into the sand at the bottom by means of a small diameter tube which was driven in at one corner. It was thought that the water would be more likely to drive out air if it were percolating upwards rather than downwards. In addition, all the water was boiled just before injecting it, to expel the dissolved air. Thus, as the water cooled it would dissolve any tiny air bubbles still present in the soil mass.

The water level was maintained at about 1 in. above the soil surface in all the tests, the top 2 in. of sand having been scraped off before inundation.

One difficulty in the inundated sand tests was the recompaction of the sand to the proper density after each test. It was found, however, that if the sand were first dug over to a depth of 12 in. and then thoroughly puddled with a spade starting at the centre and working to the corners a fairly uniform density was obtainable and, with practice, the correct density. Sometimes, when the screed was run over the surface after the

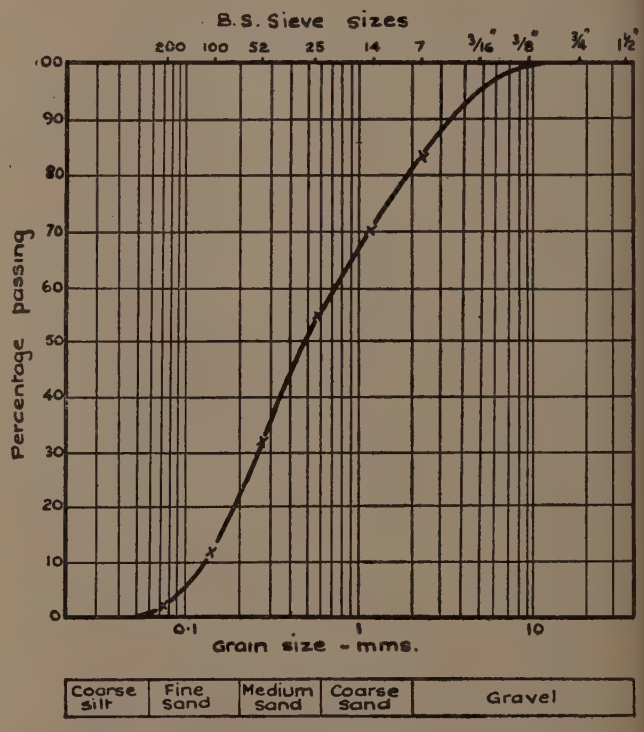


Fig. 10.—Grading of sand used in investigation

compaction, hollows would remain or possibly excessive scrapings were obtained. In these instances the sand was redug and recompacted.

Measurements of the Natural Frequency of Vibration

Footings of the following sizes were tested in turn : 10 in., 8 in., and 6 in. diameter ; 10 in., 8 in., and 6 in. square ; 18 in. × 6 in., 18 in. × 4 in., 18 in. × 3 in., 24 in. × 3 in., and 12 in. × 3 in.

Tests were made on each footing with applied dead loads between zero and about 350 lb., so that the maxi-

n pressure on the smaller foundations was of the er of 0.75 ton per sq. ft. For each dead load at it two records were taken and in the graphed results, rred to later, each point on the graph is the mean of east two test results.

After each footing had been tested the sand was dug r and recompacted to the correct density.

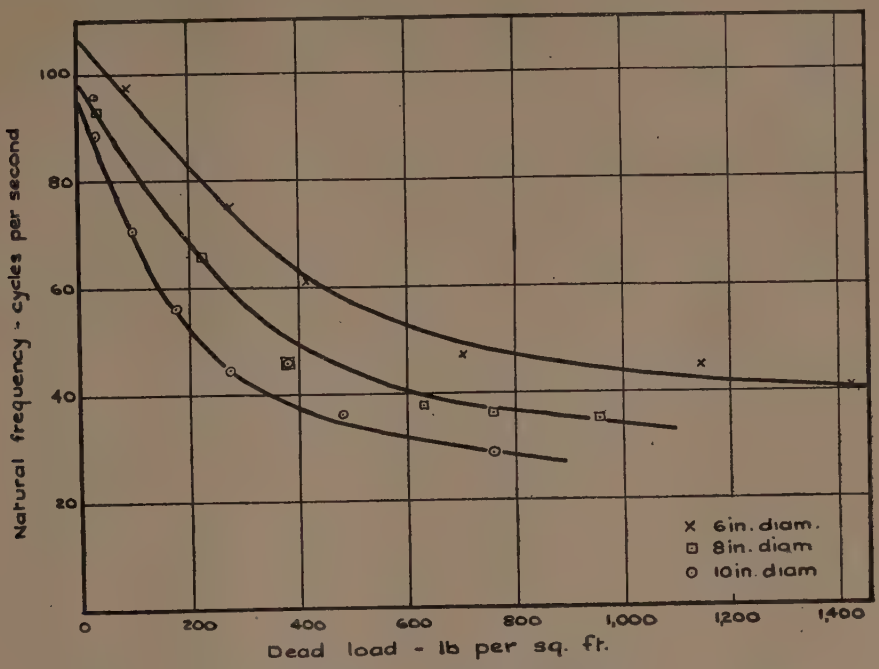


Fig. 11.—Natural frequencies of circular footings on dry sand

The relation between natural frequency and dead load er unit area for the circular footings is shown in Fig. 11. t low loads the frequencies differ by only a few per ent., but as the load is increased they diverge from ach other until, at loads in excess of about 200 lb. per q. ft., the frequency is almost inversely proportional to he footing diameter. A similar tendency is apparent or the squares and rectangles (see Figs. 12 and 13).

It will be seen too that, whatever the shape or size of he footing, the frequency at zero loading is more or less he same the value always lying between 95 and 110 ycles per second. There are two possible explanations f this. Assuming that the soil acts as a spring with a ixed node at its bottom end, it might be that for low oads the position of this node is always the same being ndependent of the footing size and shape. An alternative explanation is that, with zero load at the surface, he soil is merely in a state of internal oscillation at its own natural frequency in a similar manner to that suggested by Professor Christopherson. In this case, nstead of there being just one node, there would be a series with the soil in oscillation down to a considerable depth. According to the Christopherson theory, however, the natural frequency would be the same for a given dead load per unit area whatever the size of the foundation, which is not borne out by the experiments. It is unlikely also that his assumption of an increasing value for Young's Modulus with depth is in accordance with the conditions in these tests. Young's modulus probably would increase with depth in the case of a consolidated clay, but it should be more or less constant in a bed of sand of uniform density. Since the Christopherson theory is inadmissible the first explanation seems the more likely.

The rapid decrease in natural frequency with increasing load showed that if the spring stiffness of the ground can be assumed constant the effective mass of earth which is vibrating with the foundation is very small. An attempt was made to calculate the magnitude of this mass using the equation

$$p^2 = \frac{F}{M_s + M_t}$$

If the spring stiffness F is assumed constant then the value of the product

$$p^2(M_s + M_t)$$

must also be constant. For any two measured values of p the values of M_t, the mass of the foundation, are

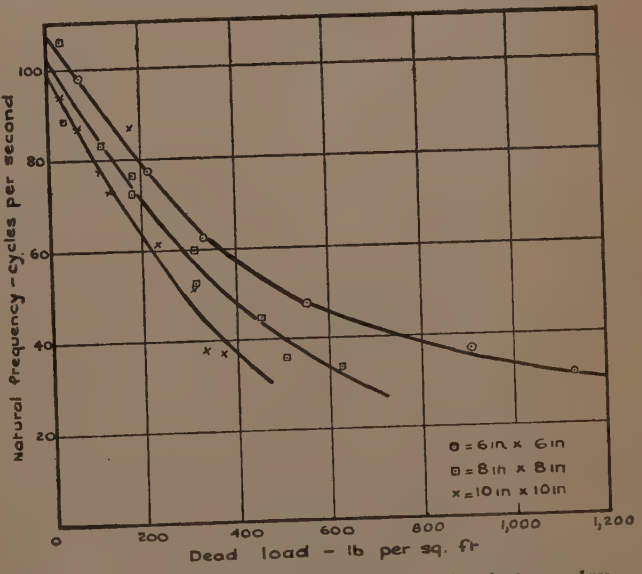


Fig. 12.—Natural frequencies of square footings on dry sand

known and equating the two values of the product gives a value for M_s, the effective mass of the soil.

Using the results of the test with the lowest loading in conjunction with the results of the test with next lowest

loading, then the next pair of test results and so on, gave the variation of M_s with the superposed load. The resulting values were graphed, a typical graph being as in Fig. 14, from which it will be seen that as the superposed dead load increases the calculated value of M_s becomes negative. The explanation is, of course, that the stiffness F is not constant but probably de-

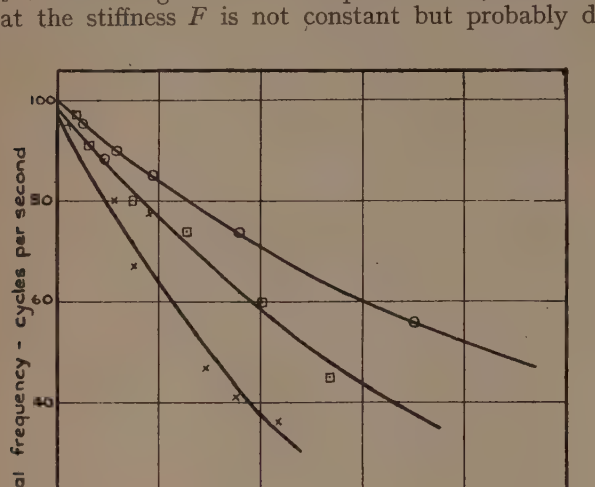


Fig. 13.—Natural frequencies for rectangles of different widths on dry sand

creases as more dead load is applied. Repeating the calculations showed that if F decreased only slightly with each increment of load the calculated value of M_s became positive.

This variation in the calculated value of M_s explains why Vios and Degebo disagreed so widely in their estimates of the ratio of M_s to M_f, and shows that it is probably quite impossible to obtain a reliable value of M_s in this way, even from full-scale test results.

The conclusion of Degebo that the resonant frequency increases with the size of the plate is also shown to be fallacious. Most investigators have accepted that this is so in recent years. The approximately hyperbolic variation with plate size obtained in the model tests may not be valid, however, for large foundations. Ordinary plate bearing tests indicate that, whilst the modulus of subgrade reaction varies more or less inversely with the plate size for small plates, the variation is not nearly so pronounced for large plates.

The two cases are not of course exactly analogous since in the plate bearing tests much of the deformation is plastic whilst the deformations under vibration must be largely elastic, but fairly similar behaviour is highly probable.

When considering the extrapolation of the model results to full scale it has to be borne in mind that the ratio of the dead load on the foundation to that of the soil which vibrates with it should be the same in model and prototype. Thus, assuming that the deformations in the ground are geometrically similar, the dead load per unit area should be scaled down proportionally to the footing width. For similarity with a foundation 10 ft. wide carrying 3 ton per sq. ft., the appropriate load for a 6 in. wide model would be only 3/20 ton per sq. ft., therefore, and smaller still for narrower models. Referring to Figs. 11, 12 and 13, it will be seen that such loads are rather below those at which the ground weight is so negligible that the frequency-plate size relatio is roughly hyperbolic.

With regard to the effect of length on the natura frequency for rectangles of the same width, Fig. shows the results of the tests on 12 in. × 3 in., 18 in. 3 in., and 24 in. × 3 in. footings superposed. It apparent that the curves for the three footings ar almost coincident, and it seems that for long narro rectangles the frequency is unaffected by a change i length provided the load per unit area is kept constant But the 6 in. square gave higher frequencies than th 18 in. × 6 in. footing (see Figs. 12 and 13).

A comparison of the results for the 6 in. diameter an 6 in. square results is given in Fig. 16. Except at lo loads the square gave lower frequencies than the circl for a given pressure, and it appears that a circle ma behave more like a square of the same mean widt rather than the same actual width. A similar tendenc though not so pronounced was found with the 8 in. an 10 in. diameter squares and circles.

One feature of the results which is perhaps surprisin is the relatively low damping. Examination of th records showed that the ratio of successive amplitude for the unloaded footings was as high as 0.95 when th amplitude was 0.0002 in. Damping is non-linear however, and in one case the ratio of successive ampli tudes was as low as 0.4 when the amplitude was 0.001 i (see Fig. 17 from which the variation in damping quite apparent). With heavy loads on the footing th vibrations were not damped nearly so rapidly since th

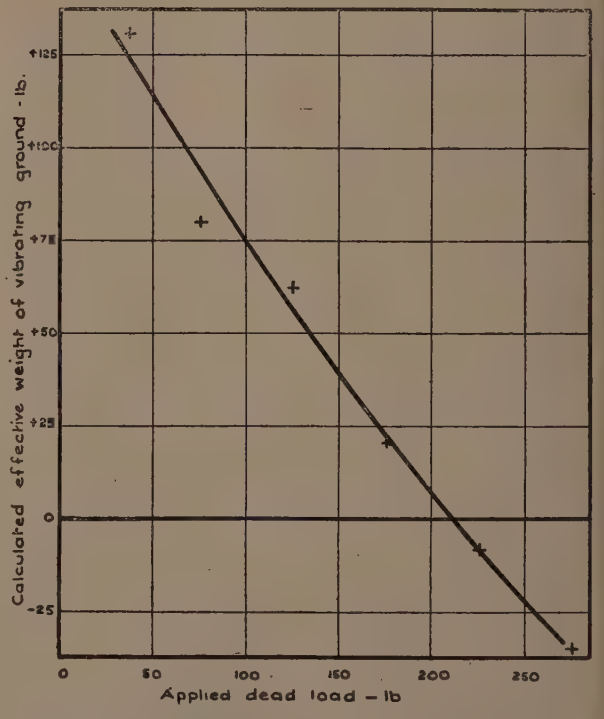


Fig. 14.—Calculated mass of soil vibrating with 12-in. × 3-in. footing subject to various dead loads on dry sand

total kinetic energy of the system is higher and th velocity generally lower.

As was pointed out by Crockett and Hammond th non-linearity of the damping was apparent from th Degebo results. When forcing the vibrations with vibrator, measurements of amplitude were taken fo various speeds, a graph similar to that in Fig. 18 bein obtained. The axis of the peak is tipped over instea of being vertical as it is when damping is linear.

The effect of inundating the foundation was next considered. It was expected that the vibrations might be much more highly damped, since the viscous losses in forcing the water from between the soil particles and drawing it back again during each cycle of vibration should be very high. In addition, the total mass of the ground plus water would be greater than that of the ground alone so that the frequency should be lowered by inundation.

The fact that the effective mass of the ground is increased by inundation has been missed by some observers who have considered the buoyancy rather

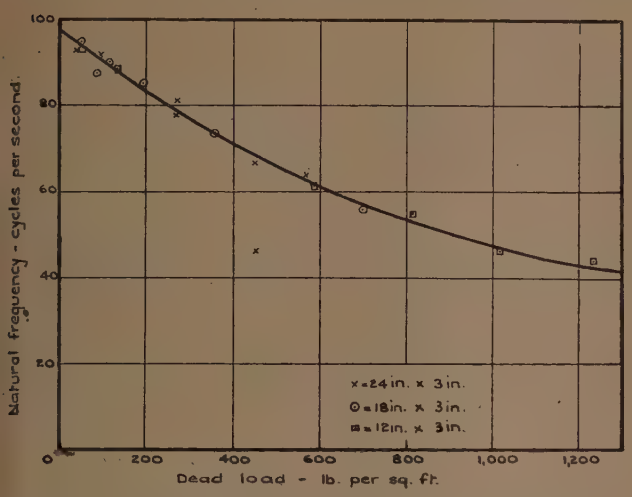


Fig. 15.—Natural frequencies of 3-in. wide footings of different lengths (dry sand)

than the inertia effects. For example, Tschebotarioff[8] has reported the case of a pumping station in which the compressors set up resonant vibrations of their foundations at the operating speed of 300 c.p.m. The water table was very close to the ground surface at that time. In an attempt to get rid of the trouble, the water level was artificially lowered by a few feet and the vibrations ceased. Tschebotarioff estimated that lowering the water table had *increased* the effective mass of the vibrating system by 7 per cent., and that this had been sufficient to alter the resonant frequency. In actual fact, lowering of the water table would *reduce* the effective mass.

The method of inundating the sand for the present tests has been described earlier. Measurement of the natural frequencies was carried out exactly as for dry sand. The measured frequencies are plotted against the applied dead load in Figs. 19, 20 and 21.

It will be seen that the natural frequencies at low loads are all considerably smaller than the corresponding ones on dry sand. At zero loading the frequencies lie between 50 and 60 cycles per second as against 95-110 on dry sand. As in the case of dry sand the frequency at zero loading varies only slightly with footing size. Another similarity with the results for dry sand is that for load intensities above about 400 lb. per sq. ft. the frequencies are almost inversely proportional to footing size. But the frequency does not tend to drop nearly so rapidly with increasing load as it does in the case of footings on dry sand (see the comparison of the results for a 12 in. × 3 in. footing in Fig. 22).

The lower frequencies, particularly at low loads, might possibly be due in part to the increased inertia of the ground because of the presence of the water. This would also be compatible with the fact that in tests involving high loads, in which the effect of the increased ground weight would be relatively small, the frequencies obtained on inundated sand are only slightly less than those for dry sand.

Alternatively, it might be that the presence of the relatively incompressible water makes the soil behave much more like a solid elastic mass. The depth to which the ground is strained during a cycle of vibration would be greater, therefore, particularly at low loads. This would reduce the spring stiffness of the ground and increase the total inertia, thus accounting for the lower frequencies. At higher loads on dry sand the depth of penetration of stress had appeared to be more like that in a purely elastic medium so that the similar frequencies when dry and inundated would be expected. The slight difference could be attributed to the extra inertia of the water.

It is probable that both the above factors play a part in producing the lower frequencies at low loads.

Although at pressures above 1000 lb. per sq. ft. the frequencies were not much lower for the inundated sand than for the dry sand, it must not be inferred that for full-scale foundations, with pressures well above this figure, there would be little difference. As was pointed out earlier when discussing the results for dry sand, the pressures on the models which correspond with those on the full scale are of the order 100-600 lb. per sq. ft. Thus it is probable that there will be considerable differences between the natural frequencies of full-scale foundations on dry and inundated sand.

An attempt was again made to calculate the effective mass of the soil vibrating with the foundation using the same technique as for dry sand. The results were again meaningless (see Fig. 23) negative values being obtained for certain loads. Despite the ground behaving more

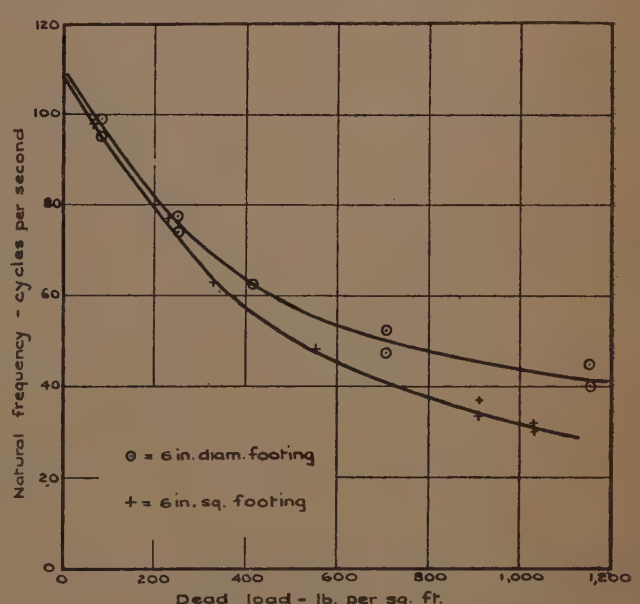


Fig. 16.—Comparison of results for 6-in. square and 6-in. diameter footings on dry sand

elastically when inundated, it is apparent that the stiffness still cannot be assumed independent of load.

As in the case of dry sand the natural frequency of long rectangles does not appear to be affected much by changes in length (see Fig. 24). But, just as for dry sand, a square gives higher frequencies than a long rectangle of the same width (see Fig. 25). Unlike the test results on dry sand, squares gave higher frequencies

than circles (see Fig. 26). No explanation of this fact is offered.

Damping was surprisingly low, and seemed to be less if anything than that for dry sand. It seems quite certain that the water does not move in and out of the interstices during a cycle, and for this reason relative movement between the soil particles and hence friction losses are probably less per unit volume of ground. This agrees with the finding described later in the paper that damping is very much less for forced vibrations when the foundation is inundated.

At this stage it was thought that some knowledge of the depth to which the soil vibrates might be obtained if a rigid boundary were introduced at different depths beneath the footing. Such tests would also indicate the manner in which the natural frequency is affected by the presence of rock strata beneath the soil on which the footing is founded.

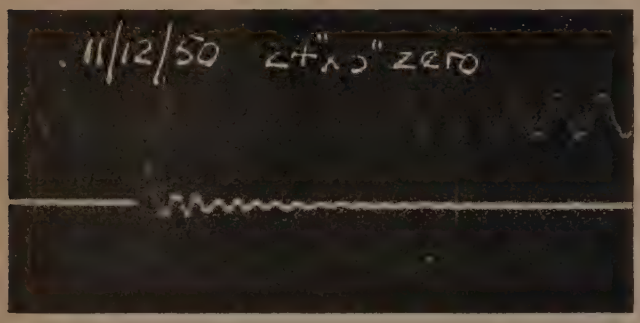


Fig. 17.—Vibration record showing variation in damping

A steel plate $\frac{3}{4}$ in. thick was used to simulate the solid boundary. Before inserting it, the top 10 in. of sand was scraped off, and the surface thus formed was carefully screeded. The plate was lowered on to this surface and hammered to ensure perfect bedding. All the sand which had been taken out of the box except for a volume equal to the volume of the plate was then replaced and recompacted to the original density.

The natural frequency was now found for a 6 in. diameter footing in the usual way, using various dead loads. One inch of sand was then scraped off, the footing replaced, and the natural frequencies again measured. This was repeated until there was only two inches of sand between the footing and the bed plate.

The frequencies obtained for various loads with different thicknesses of sand between the foundation and the steel plate are shown in Fig. 27.

The most remarkable feature of these results is that for any particular dead load the natural frequency was almost constant whatever the thickness of the supporting sand. It appears that within very wide limits the presence of a layer of rock underlying the sand has very little effect on the natural frequency. The author is unable to explain why this should be so. It would be reasonable to expect that shortening the length of the soil "spring" under the foundation thus increasing its stiffness would give higher frequencies. It may be, of course, that this would in fact have happened if the sand had been dry, and it is intended to repeat these tests at a later date using dry sand.

The Effects of Vibration on Settlement and Ultimate Load

As mentioned in the section of the paper describing the apparatus the vibrations were generated by an eccentric rotating disc mounted on a loading beam with flexible supports. The static load on the footing was obtained by jacking against the loading beam and the vibrations were transmitted through the jack to the footing, the amplitude of vibration being measured on the footing itself using the De Havilland pick-up.

Exactly as in the tests to determine the natural frequency, the top 12 in. of sand was dug up after each test and recompacted. The tests on dry sand were carried out first.

The aim of the preliminary tests was to find whether there was a critical frequency of vibration at which the footing had a much reduced bearing capacity.

In an actual foundation, one such frequency would probably be the natural frequency because, for a given driving force, the amplitude would be much greater than at any other frequency. But in the experiments the natural frequency was determined by the stiffness of the loading beam supports, not the ground, and these supports had purposely been made so stiff that it was well above the frequency of the applied vibrations. This had been done because the object of the present tests was not to investigate resonance, but what happens to the bearing properties for any given amplitude or frequency, it being immaterial whether that amplitude was due to a small vibratory force applied at resonance or a large force well away from resonance. The resonant frequency is not, of course, fixed for any one foundation but varies with the applied dead load.

It was thought possible that there might be some frequency other than the resonant one at which the bearing power might be reduced. It was conceivable, for example, that there might be one at which the soil particles might tend to rebound from each other if agitated. If so, the tests would have to be concerned with finding what determined it, as well as the effect on

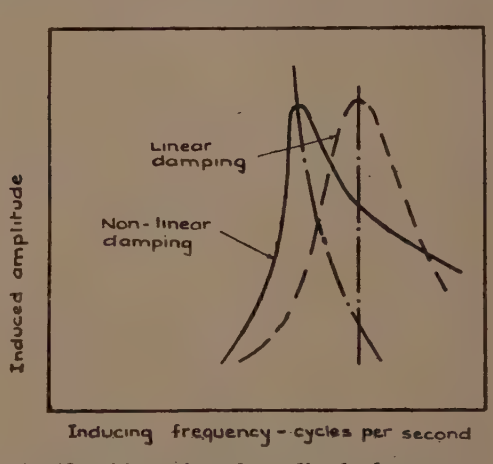


Fig. 18.—Alteration of amplitude-frequency relation when damping is non-linear

the bearing properties. If no such frequency existed, the tests would merely be concerned with the relative effects on the bearing power of different vibrations.

A footing 18 in. × 4 in. was used first, half the tests being made with static loading and half of them with vibratory loading. In these first few dynamic tests the speed of vibration was slowly increased and decreased two or three times between about 400 and 3000 cycles per minute in the course of the test. At the higher frequency the amplitude of vibration was about 0.001 in. but was much lower at the lower speeds. There did not appear to be more rapid settlement or falling off of load at any particular frequency and the ultimate

ad was only slightly lower on the average than in atic tests (see Table 1).

ABLE 1.—Comparison of Ultimate Load for 18 in. × 4 in. oting under Static and Vibratory Loading of Variable Frequency (Quick Test)

Ult. load ith vibration (tons)	Ult. load without vibration(tons)	Settlement at failure	Distance out of failure surface
—	1.11	1⅜ in.	20 in.
0.96	—	1⅜ in.	20½ in.
—	1.16	1½ in.	20¼ in.
1.135	—	1⅜ in.	17¼ in.
—	0.925	1½ in.	16 in.
1.08	—	1⅜ in.	18½ in.

verage = 1.06 Average = 1.07

Although these tests showed that there is no particular requency at which small vibrations produce a *marked* ecrease in the ultimate load it was still possible that the ecrease would be more at high speeds than at low for a est carried out in a given time. Twenty further tests ere therefore made each with a constant speed of ibration using an 18 in. × 3 in. footing. The amplitude eing 0.001 in. each time. The footings were jacked nto the sand at a rate of 2 in. per minute so that each est took about three-quarters of a minute. Ultimate oads are given in Table 2.

ABLE 2.—Ultimate Loads for Tests on 18 in. × 3 in. Footing
Amplitude = 0.001 in.

Test No.	Ultimate load (tons)		Vibration Frequency (Cycles per minute)
	With Vibration	No Vibration	
1	—	0.625	—
2	0.65	—	510
3	0.62	—	510
4	0.59	—	890
5	0.62	—	890
6	—	0.67	—
7	0.69	—	1160
8	0.63	—	1160
9	0.55	—	1480
10	0.61	—	1480
11	—	0.625	—
12	0.715	—	1820
13	0.60	—	1820
14	0.64	—	2240
15	0.60	—	2240
16	0.54	—	2560
17	0.56	—	2880
18	0.74	—	2880
19	—	0.775	—
20	0.65	—	2960
eans	0.625	0.675	

It can be seen that there was no marked drop in earing power in any particular range of frequency etween 500 and 3000 cycles per minute, and the mean ad with vibration was only 7½ per cent. lower than ithout.

In each of the above tests the load had been increased latively rapidly whilst vibrations were continuously plied. In the case of an actual foundation it is more kely that a constant dead load will be applied equal to me fraction of the static ultimate load and will remain nsibly unaltered during any subsequent vibrations. similar procedure was followed in the next tests. A fferent fraction of the static ultimate load was first of l applied in each test, and this load was kept constant whilst vibrations were superposed, the vibrations being maintained until either ultimate failure occurred or alternatively the footing had sunk into the sand a distance equal to half its width. Measurements of settlement with time were taken and plotted.

Careful thought was given to the choice of vibration frequency and amplitude. It was decided that they should be as severe as is normally met with in actual structures, but there was little information available to show what the actual value should be. There are of course two possible criteria of the maximum tolerable vibration in any building. The first of these is the comfort and well-being of the inhabitants. Reiher and Meister[11] have investigated this problem. They subjected a number of people to vibrations of various

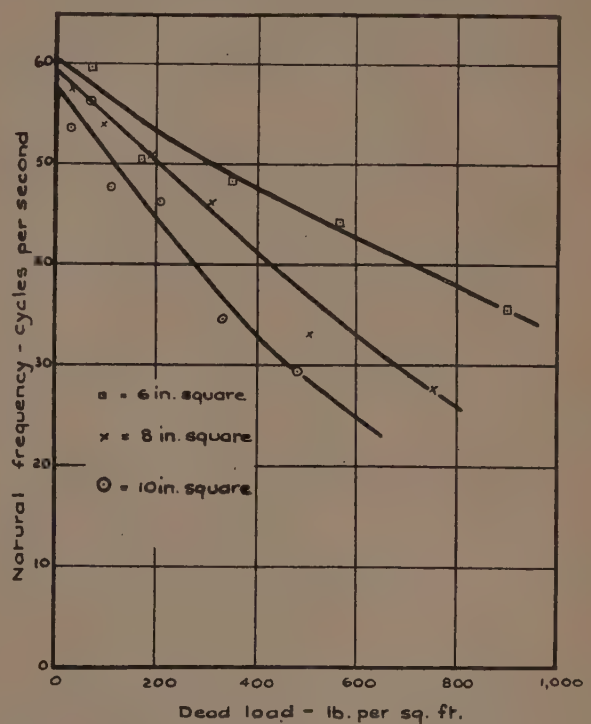


Fig. 19.—Natural frequencies of square footings on inundated sand

frequencies, amplitudes and directions, whilst standing or lying on a large vibration table. The physiological effects varied with the direction of the axis of the body relative to the direction of movement, but the results with the subject standing and subject to vertical vibration are shown in Fig. 28. The most severe vibrations which any engineer would dare inflict on the inhabitants of a building is probably given by zone 5. Another criterion of the severity of the vibrations is their effect on the structure. As was stated earlier there is some disagreement between various investigators on this point but if for simplicity the U.S. Bureau of Mines value of an acceleration of $\frac{1}{10}\,g$ is taken as the allowable limit if minor structural damage is to be avoided it will be seen that it too comes in Reiher and Meister's zone 5.

This accidental coincidence of the two criteria suggested that vibrations located in zone 5 and to the right of line A should be used in studying the long-term effects of vibration.

The next question was whether to use high frequency or low frequency vibrations. Assuming that the settlements in a given time would be the same for a high frequency—small amplitude point on line *A* of Fig. 28

as for a low frequency—high amplitude point, neither has any advantage over the other. Relatively short exploratory tests were therefore made of about one hour duration, which indicated that the high frequency—low amplitude vibrations gave more settlement in a given time, although rather less for a given number of cycles. Since Tschebotarioff also appears to have implied, although he did not specifically state, that settlement is governed by the number of applications of load irrespective of frequency for a given acceleration, it appeared that the behaviour of the footings would be determined more quickly by using a high-frequency-small amplitude oscillation. Accordingly, an arbitrary frequency of 2,400 per minute at 0.0006 in. amplitude was chosen. This frequency is less than many which occur in practice, but it is unlikely that *severe* vibrations of higher frequency ever occur since the resonant frequency of most foundations will be well below 40 cycles per second.

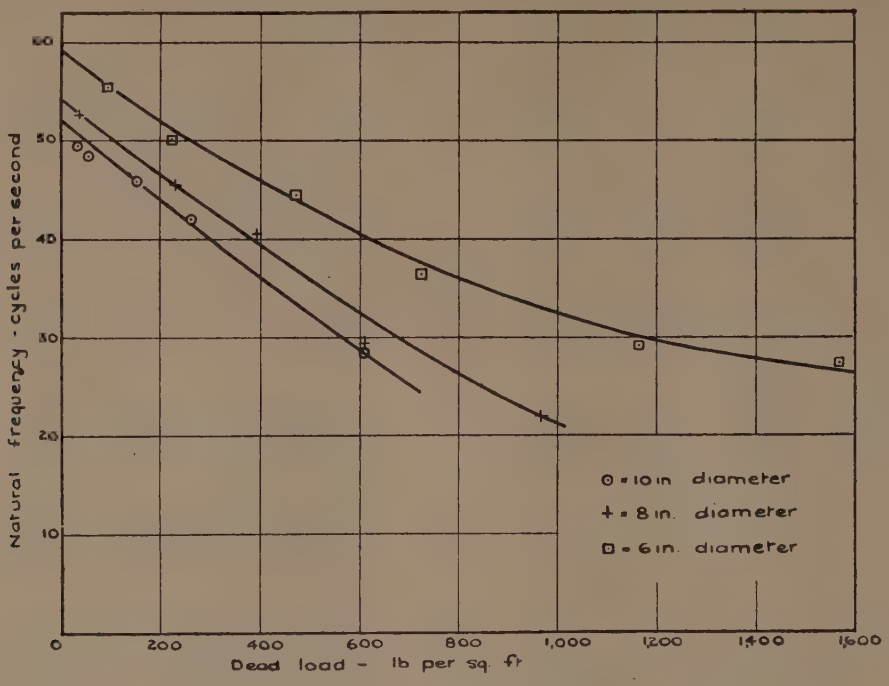


Fig. 20.—Natural frequencies of circular footings on inundated sand

The steady loads on which the vibrations were superposed were 47, 58, 69 and 80 per cent. of the static failing load (taken as the mean of six values). With each of these loads the settlement of the foundation to the nearest 0.01 in. was recorded at appropriate time

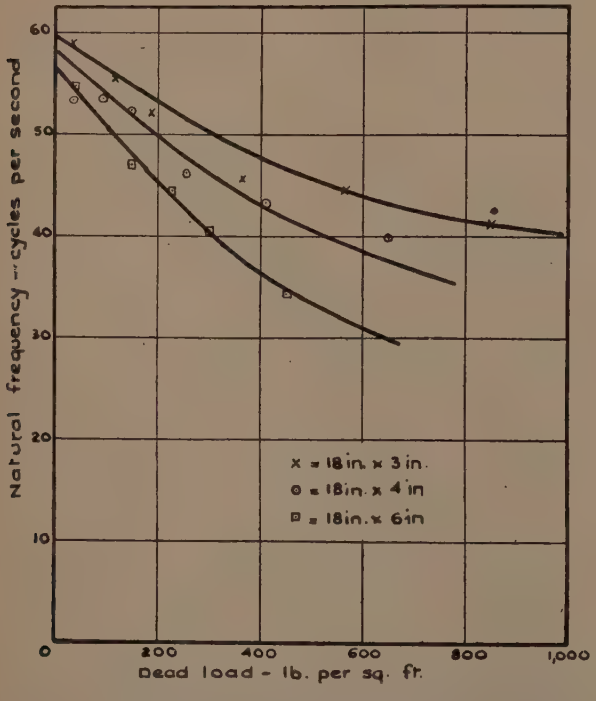


Fig. 21.—Natural frequencies of rectangles of different widths on inundated sand

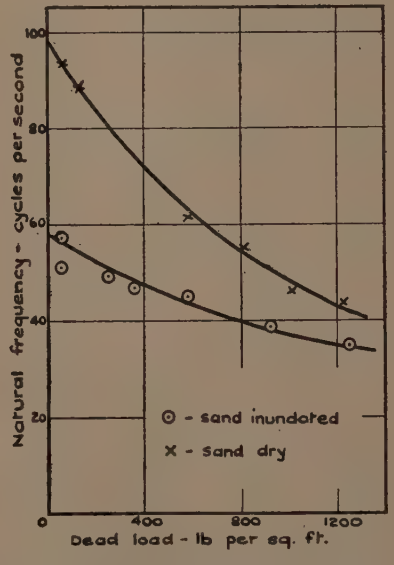


Fig. 22.—Comparison of results for 12-in. × 3-in. footing on dry and inundated sand

The results plotted semi-logarithmically. It can be seen that the superposition of the vibrations caused the footing to fail at as low as 58 per cent. of the average static failing load after about 2×10^6 cycles of loading. Although in this test a definite sliding surface was formed the footing was still capable of supporting its load provided it was

.ven into the sand rapidly. This was unusual since normal static tests the bearing power of the footing ɔps quite suddenly as soon as a definite surface of ding is formed. The failure was also unusual in that

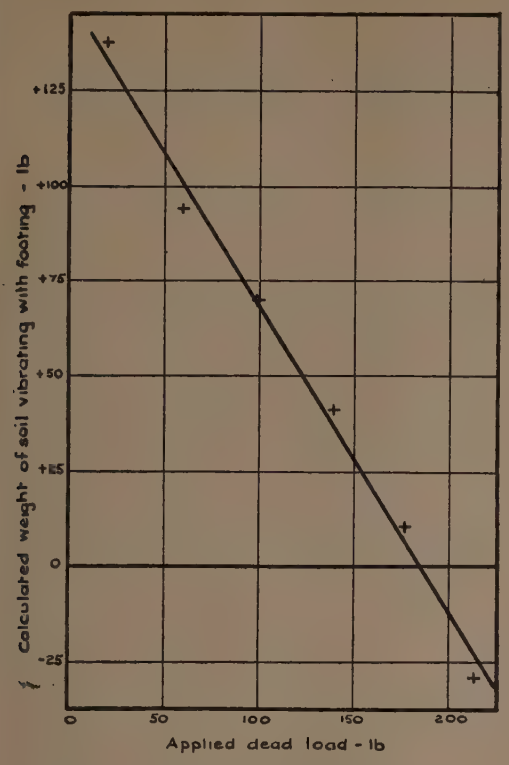
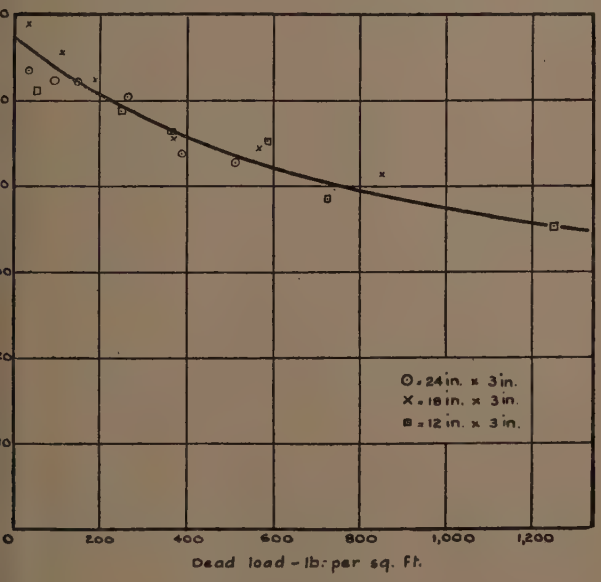


Fig. 23.—Calculated mass of soil vibrating with 6-in. diameter footing subject to various dead loads on inundated sand

distance *DC* of Fig. 30 was only 12½ in. instead of usual 20 in. average for static tests. It appeared, erefore, that the mechanism of failure was somewhat odified. With mean loads of 69 and 80 per cent. of



ig. 24.—Natural frequencies of 3-in. wide footings of different lengths (inundated sand)

static failing load the number of cycles of vibration ore failure was approximately 550,000 and 9,000 pectively. In each of these cases the mechanism failure appeared to be normal the failure surface outcropping 20 in. away from the footing and the bearing power of the footing dropping sharply immediately failure occurred. In the test with only 47 per cent. of the ultimate static load, no failure had occurred after 12 × 10^6 cycles of vibration, but the settlement was still continuing at a steady rate of about 0.13 in. per million cycles. Thus, although the footing might be capable of supporting 47 per cent. of the static load indefinitely, settlement would certainly become excessive before long.

It will be seen from Fig. 29 that the portion of the settlement at failure which was due to vibration increased as the mean footing load decreased and the time of vibration increased. But there is not so much difference if the settlements which occurred whilst applying the initial loads are added to those which occurred during

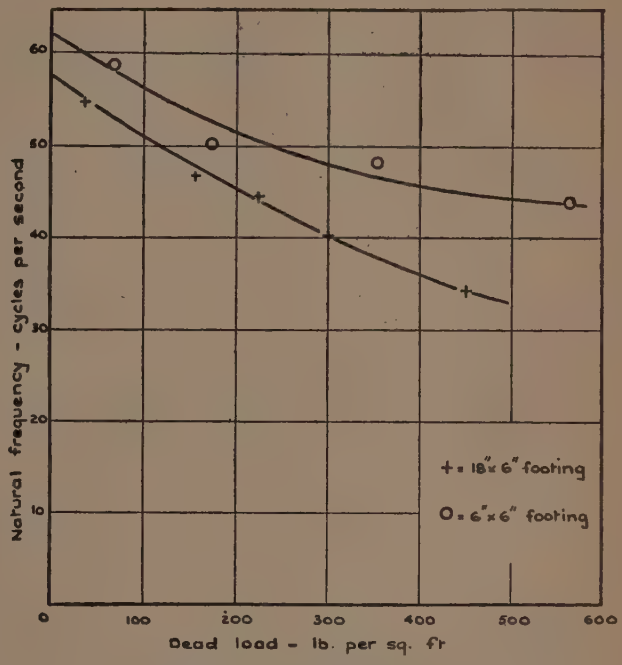


Fig. 25.—Comparison of frequencies for 18-in. × 6-in. and 6-in. square footings on inundated sand

vibration. The sum of the two settlements is shown in Table 3.

TABLE 3.—Table of Settlements obtained on Tests involving Constant Mean Loads with Vibrations Superposed

Footing size 18 in. × 4 in.

Load — % of static Ultimate	Settlement before vibration	Settlement during vibration	Total Settlement	Remarks
100	1.54 in.	zero	1.54 in.	Static test
80	0.47 in.	0.96 in.	1.43 in.	Failure under vibration
69	0.36 in.	1.14 in.	1.50 in.	
58	0.23 in.	1.28 in.	1.51 in.	
47	0.19 in.	1.52 in.	1.71 in.	No failure

These tests involving lengthy vibration could not be run continuously as this would have involved running 24 hours per day for several days (the longest test involved vibration for 5000 minutes). The vibrations were therefore stopped overnight and in one case over two week-ends. It is not thought that this affected the results in any way since no discontinuity was detectable after these idle periods.

In the tests on a 12 in. × 3 in. footing similar results were obtained to those shown in Fig. 28. An actual shear failure was obtained with only 55 per cent. of the mean static failing load after 2.5 × 10^6 vibrations, but with a mean load 45 per cent. of the static failing load

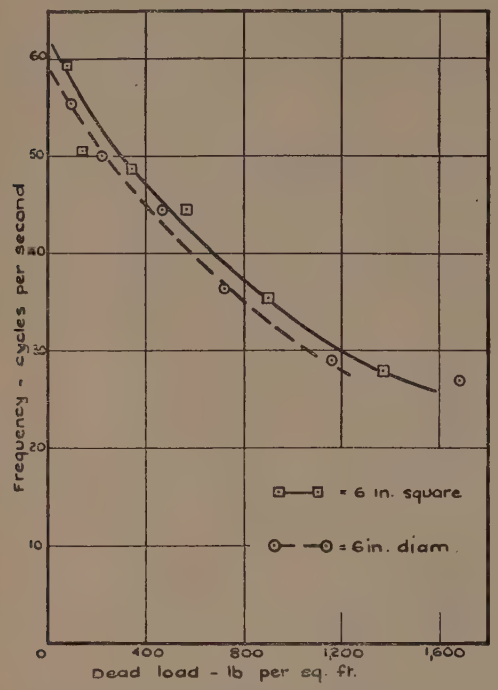


Fig. 26.—Comparison of frequencies for 6-in. circle and 6-in. square on inundated sand

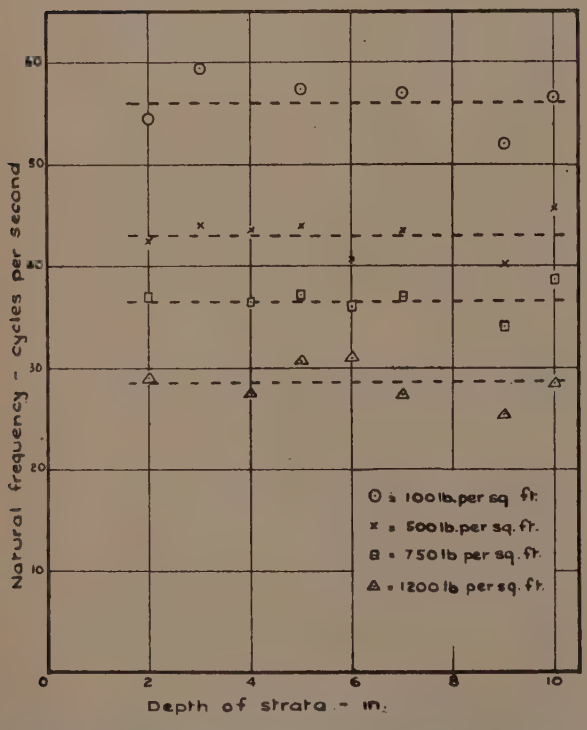


Fig. 27.—Frequencies obtained for different dead loads with a layer of solid strata at different depths beneath a 6-in. diameter footing (sand inundated)

ultimate failure was not reoched after 10 × 10^6 cycles of vibration, but the settlement was already 25 per cent. greater than that obtained at failure in an ordinary static test. The settlement values at other loads were similar to those in Table 3.

Similar tests to the above were later repeated on inundated sand. The first tests which were made were static ones on an 18 in. × 4 in. footing to determine the failing load without vibration. The footing was driven into the sand by turning the jack one tooth every 1¼ minutes. This represented an average rate of penetration of 1 in. in 250 minutes although it was actually applied in increments of 0.005 in. The mean value of the ultimate load was 0.83 tons. In one test a check was made that the ultimate load was not being increased by the test being carried out too quickly thus setting up seepage stresses in the pore water which would help to resist failure. The footing was driven

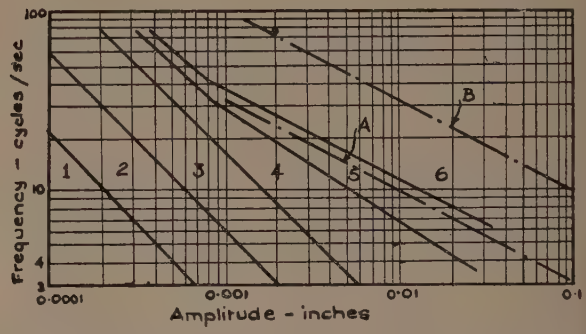


Fig. 28.—Zones of response of standing humans to different intensities of vertical vibration. (After Reiher and Meister)

Zone	Response
1	Not perceptible.
2	Slightly perceptible.
3	Distinctly perceptible.
4	Strongly perceptible.
5	Disturbing—detrimental effects, e.g., severe headaches if prolonged.
6	Very disturbing—may have detrimental effects if of short duration.

Line A—acceleration $= \frac{1}{10} g$.

Line B—acceleration $= g$.
Zone A—B some plaster cracking.
Beyond B—Structural damage may be severe.

into the sand at the same rate as in the other tests until the load reached 0.78 tons. The test was then discontinued for two days during which time the load fell to 0.76 tons. On resuming the test the ultimate load was 0.82 tons. It seems reasonable to infer from this that the rate of penetration was sufficiently slow to have little effect on the ultimate load.

A test involving vibration was now tried. It was immediately apparent that the amplitude of oscillation of the footing was approximately double that obtained with the same out of balance mass on the dry sand. With this large amplitude the footing was almost incapable of supporting load and settled as fast as the jack could be screwed with only about 20 per cent. of the static failing load of 0.83 tons applied.

The high amplitude shows that damping is much smaller for the inundated sand than for the dry sand. The exact ratio could not be determined because part of the damping of the system is provided by the rubber supports of the loading beam. This test confirms the finding in the experiments concerned with the effect of inundation on the natural frequency that the inundated soil mass acts somewhat like a solid bed of rock through which stress waves are readily transmitted with little damping.

Regarding the loss of bearing power it is possible that ause of viscous stresses in the water the sand under footing cannot recover quickly when the footing is ving upwards so that the footing actually leaves it

load, and no failure surface were formed. With a mean load of 60 per cent. of the static ultimate, failure occurred in only 7½ minutes at 1.4 in. settlement (after commencing vibration) whilst at 75 per cent. of the static

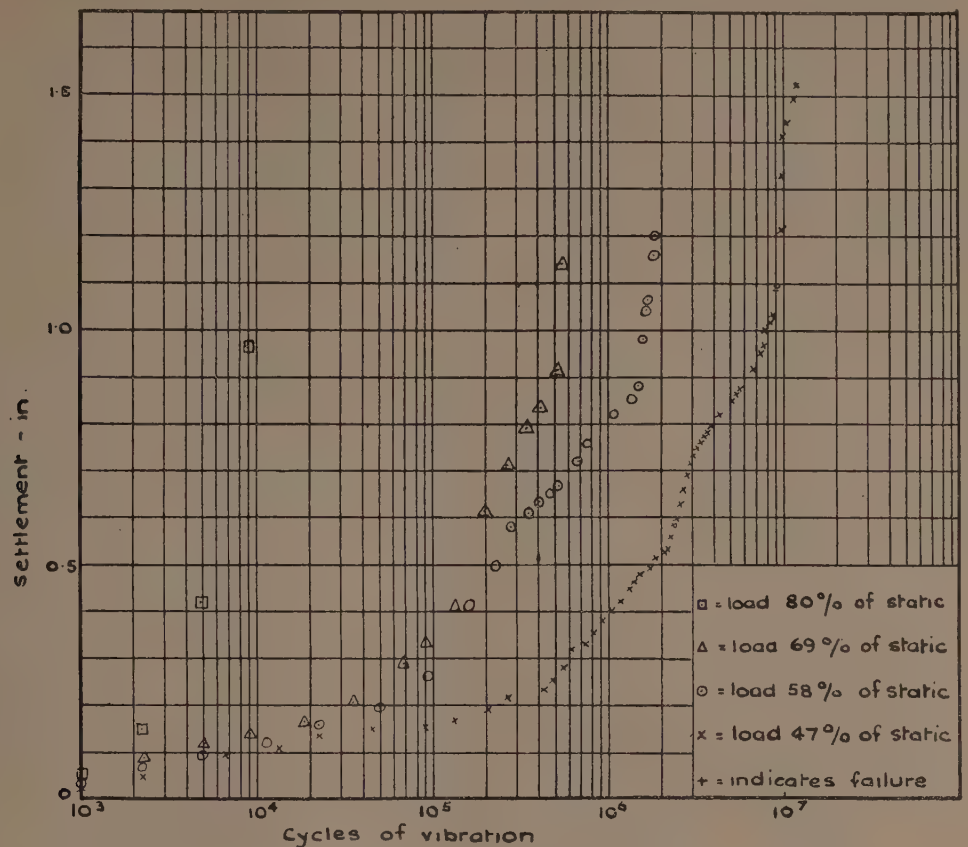
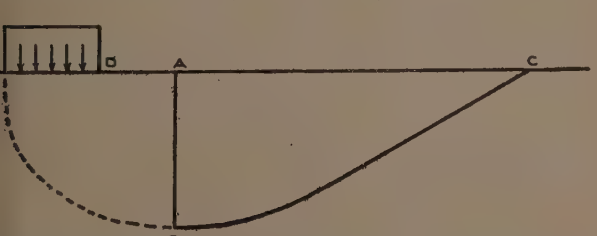


Fig. 29.—Settlements with different percentages of the static ultimate load under the action of severe vibration (dry sand)

ind, meeting it again on the downward stroke. The nmer blow effect could reasonably be expected to uce the bearing power.

The out of balance mass was now reduced approately to give the same amplitude as in the dry tests. ration tests were then made with constant mean ds of 0.62, 0.50, 0.38, 0.26 and 0.14 tons.

The time settlement curves are shown in Fig. 30. It be seen that at only 0.14 tons (17 per cent. of the



ig. 30.—Mechanism of failure for footing on sand. Vedge ABC slides up BC. Sand in ABE follows wedge allowing footing to settle

tic ultimate load) settlement was continuous and ly rapid, the settlement reaching 2 in. after about < 10^5 cycles (180 minutes approximately) and was increasing at the rate of 1 in. in 2¼ hours. At per cent. of the static load settlement was about ce as rapid as with 17 per cent., whilst with 46 per t. the settlement was 2 in. in 40 minutes.

But although the settlement was so great in these s, the footings were still capable of maintaining their

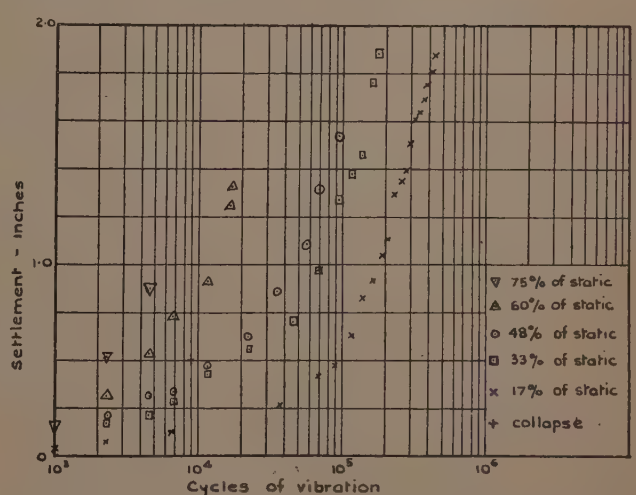


Fig. 31.—Settlements with different percentages of the static ultimate load under the action of severe vibration (inundated sand)

failing load vibration caused failure in only two minutes at a settlement of only 0.88 in. The total settlements including that whilst applying the mean load before beginning vibrations were 1.65 and 1.1 in. respectively.

Conclusions

Although it would be unsafe to try and extrapolate the behaviour of full size foundations from the results of the

present tests it is likely that the results would be qualitatively similar.

For the small size foundations the following conclusions can be drawn.

(a) Regarding the natural frequency of vertical oscillation.

(i) For very lightly loaded footings the natural frequency is almost independent of the footing size for a given load intensity, but for high loads it is almost inversely proportional to the foundation width. For dead load intensities which are comparable with those on actual foundations the behaviour is somewhere between these two extremes.

(ii) It is not possible to estimate the effective volume of ground which acts in the vibration system since the volume appears to vary. The calculated value is also distorted by the fact that the spring stiffness of the ground also varies.

(iii) For long rectangles of equal width the natural frequency appears to be independent of the length for a given load intensity. Squares give higher frequencies than rectangles of the same width.

(iv) Square footings had slightly lower natural frequencies than circles of the same width. It may be that a circular footing has approximately the same frequency as a square of the same mean width.

(v) For inundated foundations the natural frequencies were again almost independent of the footing shape at low load intensities, but the frequencies were only a little over half those on the dry sand. At high loads the frequency was again roughly inversely proportional to the footing width but was not nearly so low relative to the value for a corresponding footing on dry sand as it was at low load intensities.

(vi) After inundation the natural frequency was again independent of length for long rectangles, but squares gave higher frequencies than circles.

(vii) The introduction of a rigid layer of strata at different depths beneath a circular footing diameter d showed that for any particular load on the footing the natural frequency was almost constant whatever the depth of the rigid layer between the limits $d/3$ and $2d$.

(b) Regarding the effect of vibrations on bearing power and settlement.

(i) Vertical vibrations in structures which would be severe enough to cause slight structural damage and produce adverse physiological effects on people have negligible effect on the ultimate load of a foundation on dry sand provided they are only applied for a short period of time—say less than one minute.

(ii) For a given amplitude of oscillation there does not appear to be any particular frequency at which the effects of vibration are most pronounced. There is of course greater possibility of large amplitude oscillations if the driving force is applied at approximately the natural frequency.

(iii) If severe vibrations are continued for a sufficiently long time the foundation may fail at a smaller load than the ultimate for static loading. It appears that for a footing on dry sand subjected to severe vibration, a normal failure can occur at about 50 per cent. of the static failing load after rather more than 10×10^6 cycles. For mean loads greater than 50 per cent. of the static ultimate load failure took place after appropriately less cycles of loading.

(iv) Although there was no failure in the ordinary sense of the word with loads slightly less than 50 per cent. of the static ultimate, settlement was so great as to amount to failure after about 12×10^6 cycles of vibration.

(v) Inundation reduces damping effects. Thus a given impressed force will tend to cause greater vibrations than on dry sand, although the exuberance will also depend largely in each case on the incidence of resonance.

(vi) Vibration of the inundated foundations gave similar but more marked reductions in bearing power compared with those on dry sand. Failure occurred at much lower fractions of the static ultimate load after a shorter period of vibration.

Acknowledgements

The work described in this paper was carried out in the Engineering Department of the University of Aberdeen with the active encouragement of Professor Jack Allen, D.Sc., M.I.C.E. The author was helped in much of the experimental work by two final year students, Ian Davidson, B.Sc., and R. A. K. Long, B.Sc.

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Discussion

The Literature Committee would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be received by June 1st, 1953.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of ructural Engineers was held at 11, Upper Belgrave reet, London, S.W.1, on Thursday, January 22nd, 53, at 6 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., I.Struct.E. (President), in the Chair.

The following members were elected in accordance th the Bye-Laws. Will members kindly note that the ections as tabulated below should be referred to when nsulting the Year Book for evidence of membership ?

STUDENTS

AKER, Anthony Frederick, of Barnet, Herts.
ENNETT, John Brian, of Burton-on-Trent, Staffs.
ONTOFT, Anthony John, of Scunthorpe, Lincs.
ARTER, Brian Duke, of Port Elizabeth, South Africa.
E QUERVAIN, Emil Hugh, B.Sc.(Eng.) London, of Woking, Surrey.
LLERINGTON, Charles Richard, of Scunthorpe, Lincs.
LLIS, William George, of Berkhamsted, Herts.
ELLOWS, Brian, of Leeds, Yorks.
ODEY, Donald Alfred, of Ndola, Northern Rhodesia.
UNT, John, of Huddersfield, Yorks.
OWU, Charles Olatunji, of London.
ELSEY, Peter John, of London.
AKE, John Brian Francis, of Harrow, Middlesex.
ASH, Alan Blake, of Caterham, Surrey.
URNELL, Raymond William, of Bristol.
EWART, Alan Alfred, of Gateshead, Co. Durham.
UART, Peter Leslie, of London.
MMERBELL, George Bernard, of Newcastle upon Tyne.
DENHAM, Richard Sidney, of Ilford, Essex.
CKERS, Rex Adrian, of London.
ATLING, Anthony John, of London.
ATT, David, of Uddingston, Lanarkshire.

GRADUATES

MERY, Montague William, of Birmingham.
NDER, Paul Frank, of Skewen, Glam.
LACK, William Angus, of Dagenham, Essex.
MIS, Roy Alfred, of Romford, Essex.
SWELL, William, of Stoke-on-Trent.
URROSE, George Arthur, of Liverpool.
OO YIN CHEE, B.Sc.(Eng.) Hong Kong, of Ipoh, Malaya.
ORD, Keith Alan, of Morden, Surrey.
REENE, Dennis Leslie, of Northolt, Middlesex.
NSHELWOOD, William Hutton, of Glasgow.
ORN, Cyril Desmond, of Bridgend, Glam.
ORTHGREAVES, Kenneth Roy, of Hampton, Middlesex.
RMAR, Pyara Singh, of Nairobi, Kenya.
TANKAR, Prabhakar Ganesh, B.E.(Civil) Bombay, of Bombay, India.
DGEON, Norman William, of Woking, Surrey.
OBINSON, William Whitfield, of Liverpool.
HOLNICK, Gabriel, B.Sc.(Eng.) Cape Town, of London.
MPSON, Frank, of London.
URGEON, James Harold, of Derby.
NSEN, John Frederick, B.Sc.(Eng.) South Africa, of Johannesburg, South Africa.
EHRLE, John Emile Charles, of Stanmore, Middlesex.
HITESMITH, Noel Gilbert, of Scunthorpe, Lincs.

MEMBER

EILER, Emil Lawrence Anthony, of Newcastle upon Tyne.

TRANSFERS

Students to Graduates

DURLEY, John Edward Charles, of Loughton, Essex.
MARTIN, Geoffrey Walter, of London.
PENDAL, Bryan James, B.Sc.(Eng.) London, of Berkhamsted, Herts.

Graduates to Associate-Members

COHEN, Stanley Ernest, B.Sc.(Eng.) Rand, of Johannesburg, South Africa.
LEVER, David Litler, of Bolton, Lancs.
WILLIAMSON, James Wilson, B.Sc.(Eng.) Edinburgh, of Toronto, Canada.

Associate-Member to Member

MACLEAN, Neil Gordon, of Burgess Hill, Sussex.

Members to Retired Members

BANKS, Charles Noake, of Lymington, Hants.
FOSTER, Harold Pearson, of Leeds.
HILL, Frederick, M.M., of Chesterfield, Derbyshire.

OBITUARY

The Council regret to announce the death of ARNOLD ORMOND (Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of JOHN WILLIAM DUNN, WILLIAM ADAM SINGLETON (Associates) ; ROBERT SPENCE (Associate-Member) ; EDWARD MERVYN DENCH, WILLIAM DOUGLAS KITCHING (Graduates) ; ROBIN SALMON (Student).

EXAMINATIONS

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 14th and 15th, 1953 (Graduateship), and July 16th and 17th (Associate-Membership).

ANNUAL DINNER

The Annual Dinner will be held at the Dorchester Hotel, Park Lane, London, W., on Friday, May 1st, 1953, at 7 o'clock for 7.30 p.m. Speeches will conclude at approximately 9.30 p.m., and will be followed by dancing until 2 a.m. Particulars have been circulated to members.

SUMMER MEETING

The first Summer Meeting of the Institution to be held since 1939 will take place in Scotland from May 20th to 22nd, 1953. The Headquarters will be at the Royal Technical College, Glasgow, and members will have an opportunity of enjoying the hospitality of the Scottish Branch for certain items in the programme, particulars of which were circulated with the February issue of the Journal. Members who wish to participate and who have not yet applied for the requisite application forms are urged to write to the Secretary without delay.

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :

ROYAL SANITARY INSTITUTE HEALTH CONGRESS, HASTINGS, APRIL, 1953
Mr. J. E. Swindlehurst, O.B.E. (Past President).

British Standards Institution Committee on Arc Welding as applied to Tubular Steel Structural Members

Mr. H. A. Cadwell (Member).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, March 12th, 1953

Joint Meeting with the Institute of Welding, at 6 p.m., when Mr. E. M. Lewis will give a paper on "The Construction of the new Testing Laboratory of the British Welding Research Association at Abington."

Thursday, March 26th, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Professor A. G. Pugsley, O.B.E., F.R.S., D.Sc., M.I.C.E., F.R.Ae.S., (Vice-President), will give a paper on "A Simple Theory of Suspension Bridges."

Wednesday, April 15th, 1953

By courtesy of the British Section of the Société des Ingénieurs Civils de France, members of the Institution are invited to attend a meeting at the Iron and Steel Institute, 4, Grosvenor Gardens, London, S.W.1, on Wednesday, 15th April, at 6 p.m., when Monsieur A. Lazard will give a paper entitled "Flexual Plasticity in Mild Steel."

Thursday, April 16th, 1953

An additional Ordinary Meeting will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, April 16th, 1953, at 6 p.m., when a paper on "Economy in the use of Structural Materials" will be given by Mr. G. A. Gardner, O.B.E., M.I.Struct.E., Chief Structural Engineer, Ministry of Works, Mr. R. Morton, B.Eng., M.I.C.E., Deputy Civil Engineer-in-Chief, Admiralty, and Mr. L. R. Creasy, B.Sc., M.I.C.E., Deputy Chief Structural Engineer, Ministry of Works.

Thursday, April 23rd, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. J. H. A. Crockett, B.Sc., A.C.G.I., D.I.C., A.M.I.C.E., (Associate-Member), and Mr. D. B. O'Neill, B.Sc., will give a paper on "Modern Machine Foundation Technique."

Thursday, May 14th, 1953

An additional Ordinary Meeting will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 14th, 1953, at 6 p.m., when Mr. G. M. Frost, M.I.Struct.E., will give a paper on "The Structural Aspect of Power Station Design in South Africa."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

Thursday, May 28th, 1953

Ordinary General Meeting for the election of members, 5 p.m.

ANNUAL GENERAL MEETING

The Annual General Meeting of the Institution of Structural Engineers will be held at the Royal Technical College, George Street, Glasgow, on Thursday, May 21st, 1953, at 5.30 p.m.

BENEVOLENT FUND

The Annual General Meeting of the Voting Contributors to the Institution of Structural Engineers' Benevolent Fund will be held at the Royal Technical College, George Street, Glasgow, on Thursday, May 21st, 1953, at 5.45 p.m.

HONOURS AND AWARDS

In offering their sincere congratulations to the following members on the distinctions recently conferred upon them, the Council feel that they are also expressing the good wishes of the Institution.

Order of the British Empire—C.B.E.

Mr. H. V. Lobb (Associate)

Order of the British Empire—M.B.E.

Mr. H. H. Leys (Associate-Member)

Honorary Degree of Doctor of Science, Leeds University

Professor J. F. Baker, O.B.E. (Member)

Honorary Degree of Master of Science, Leeds University

Mr. W. Hunter Rose (Member)

PERSONAL

Mr. Vincent Burr (Associate) announces that he has taken into partnership his two Associated Architects, Mr. Edmund G. Harker and Mr. John J. Hickie. The practice will continue to be carried on at 85, Gower Street, London, W.C.1, under the title "Vincent Burr and Partners."

JOURNAL CASES AND BINDING, 1952

A binding case can be supplied for the twelve issues of the Journal, January-December, 1952 (Vol. 30), price 11s. 6d. post free. The price for binding volumes is 26s. 6d. per volume, inclusive. This is for the half-leather binding which has been in use for some years.

It is requested that all parcels and Journals forwarded for binding should bear the name, address and rank of the member concerned. All volumes for binding must be despatched to the Institution by March 31st, 1953.

An Index will be included in all volumes bound. This Index will not be generally distributed, but members and others wishing to have a copy should apply to the Secretary.

DRURY MEDAL AWARD

The fourth competition for the above award will take place in 1953. The subject is the design of the structure of a new factory building. The material of construction is entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of structural arrangement. Economy in the use of steel is an important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary; envelopes to be marked in the top left-hand corner, "Drury Medal Award."

The closing date for the competition is October 1st, 1953.

The general conditions of the competition are as follows:—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted

ll be countersigned by a corporate member of the titution or, failing this, shall be accompanied by a laration on a prescribed form signed by the candidate the presence of a Justice of the Peace or a Commis-ner for Oaths.

MACLACHLAN LECTURE COMPETITION

The closing date for the receipt of entries for the cLachlan Lecture Competition is Tuesday, March 31st, 53. Particulars of the competition are as follows :—

1. The Institution of Structural Engineers shall stitute a written lecture to be known as the MacLachlan cture, and to be held annually.

2. The subject of the Lecture may be on any aspect of ructural Engineering so long as in every second year e subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture all be confined to Associate Members of the Institution, o are under the age of 32 years.

4. All papers entered for the competition shall be bmitted to assessors to be appointed by the Council the Institution, and all such papers (including the ize-winning Lecture) shall be available for publication the Journal of the Institution at the discretion of the uncil.

5. No paper submitted shall have been published or ad elsewhere.

6. The winner of the competition shall be required to esent the Lecture to a meeting of the Institution at ich he will be presented with the sum of £17 10s. 0d.

7. Should a competitor's paper be considered worthy ranking second in merit he will receive a consolation ard of £5.

8. In the event of there being no winner of the mpetition in any one or more years, whether because lecture submitted is considered to be of sufficient erit to warrant award, or for any other reason, the stitution shall transfer the above sums to the Research und of the Institution.

Particulars of the Competition for 1953

1. The MacLachlan Lecture will be given at a meeting the Institution to be arranged towards the end of 1953.

2. The subject of the Lecture shall be confined to eel structures.

3. The work should be submitted as the script of a cture which the author, if successful in the competition, ll deliver before an audience in the course of about ne hour. The development of mathematical formulæ nd detailed calculations should be avoided as far as ssible in the text; if they are essential they should be nbodied in appendices. Photographs, drawings, graphs, c., which would appear as illustrations to the lecture in ublished form, should accompany the script. If dditional illustrations would be shown as slides, a list these should be included.

4. Six copies of each Lecture should be submitted and ould be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the stitution is Tuesday, March 31st, 1953.

ONDON GRADUATES' AND STUDENTS' SECTION

The Annual General Meeting of the Section will be eld at 11, Upper Belgrave Street, London, S.W.1, on uesday, March 10th, 1953, at 6 p.m., and will be llowed by a film lecture on "Iron and Steel" by r. W. Gibson Martin, of the British Iron and Steel ederation. It is hoped that as many members as ssible will attend the meeting.

Hon. Secretary : C. Allen Brown, 43, Coolgardie venue, Highams Park, London, E.4.

BRANCH NOTICES

Lancashire and Cheshire Branch

The following meetings have been arranged :—

Tuesday, March 3rd, 1953

Joint Meeting with the Reinforced Concrete Association. Mr. E. Shepley, B.Sc., A.M.I.C.E., on "Prestressed Concrete Framework for Liverpool University Medical School."

Tuesday, March 26th, 1953

Joint Meeting with the Institution of Civil Engineers, North-Western Association.

Wednesday, April 29th, 1953

Annual Business Meeting. Films : "British Steel" and "A Plan Takes Shape."

All meetings, unless otherwise stated, will be held in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

Midland Counties Branch

The following meetings have been arranged :—

Friday, March 27th, 1953

Mr. S. M. Cooper (Associate Member), on "The Tacoma Narrows Bridge Design Features and Collapse Investigations."

Tuesday, April 14th, 1953

Mr. V. H. Lawton (Member), on "Some Notes on Designing to B.S. 449," at the Welbeck Hotel, Nottingham, at 7.0 p.m.

Friday, April 24th, 1953

Annual General Meeting, followed by paper by Mr. O. W. Jones, B.Sc., A.M.I.C.E., (Member), on "Reinforced Concrete Foundations and Structures for Blast Furnaces and Materials and Handling Plant at Shotton, nr. Chester."

All meetings, unless otherwise stated, will be held in the James Watt Memorial Institute, Birmingham, at 6.0 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

Midland Counties Branch—Graduates' and Students' Section

The following meeting has been arranged :—

Tuesday, March 31st, 1953

Annual General Meeting, followed by talk by Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., (Past Chairman, Yorkshire Branch), on "Unusual Industrial Structures." At the James Watt Memorial Institute, Birmingham, at 6.30 p.m.

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

Northern Counties Branch

The following meetings have been arranged :—

Tuesday, March 3rd, 1953

Ladies' Guest Night at Middlesbrough.

Wednesday, March 4th, 1953

Ladies' Guest Night at Newcastle.

Tuesday, March 31st, 1953

Annual General Meeting at Middlesbrough.

All meetings commence at 6.30 p.m., the Middlesbrough Meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and those at Newcastle in the Neville Hall, near Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, March 3rd, 1953

Mr. J. C. Malcolmson, B.Sc., (Member), on "Recent Developments in Prefabricated Concrete Structures."

Tuesday, March 31st, 1953

Annual General Meeting.

All meetings, unless otherwise stated, will be held in the College of Technology, Belfast, commencing at 6.45 p.m., preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6.0 p.m.

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Wednesday, March 18th, 1953

Mr. H. Nelson, B.Sc., A.R.T.C., on "Tension Members in Engineering Structures," at Royal Technical College, George Street, Glasgow, at 6.0 p.m.

Tuesday, April 14th, 1953

Annual General Meeting at the Ca'doro Restaurant, Union Street, Glasgow, at 6.0 p.m.

Hon. Secretary: D. G. Drummond, B.Sc., M.I.Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, March 11th, 1953

Mr. J. R. M. Ewen, B.Sc., on "Tubular Structures," at Swansea.

Tuesday, March 17th, 1953

The above meeting will be repeated at Cardiff.

Friday, March 27th, 1953

Branch Annual Dinner at Swansea.

Tuesday, April 14th, 1953

Mr. J. R. M. Poole, on "Recent Developments in Timber Structures," at Cardiff.

Wednesday, May 13th, 1953

Annual General Meeting at Swansea.

Meetings in Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

The Third Meeting of the Session was held in the Geology Theatre of the University of Bristol on Friday, December 5th, 1952, when Mr. P. J. Ward's paper on "The Design and Erection of Television Masts," was read by Mr. C. H. Hill, A.K.C., A.M.I.C.E. The paper proved to be of great interest to the members and friends present. A vote of thanks was proposed by Lt.-Colonel Ward, T.D., A.M.I.Mech.E., (Associate-Member).

The Fourth Meeting of the Session was held in the Geology Lecture Theatre of the University of Bristol on Friday, January 2nd, 1953, when Mr. F. R. Bullen, B.Sc., M.I.C.E., (Member of Council), read a paper entitled "Unusual Design of a Large Constructional Shop." A vote of thanks was proposed by Mr. F. K. Fennell, (Associate-Member).

The following meetings have been arranged :—

Thursday, March 5th, 1953

Combined meeting with the Institution of Civil Engineers. Mr. L. Richardson, A.M.I.C.E. (Associate Member), on "Plymouth 'B' Power Station," at the Reception Room, University of Bristol, at 5.30 p.m. Tea at 5.0 p.m.

Friday, April 10th, 1953

Annual General Meeting, followed by film show and/or papers presented by Junior Members, in the Geology Lecture Theatre, University of Bristol, at 6.0 p.m. preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 39 Effingham Road, St. Andrew's Park, Bristol, 6.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, March 18th, 1953

Mr. H. E. Manning, B.Sc., M.I.C.E., (Member of Council), on "Report on the Papers given on Concrete Shell Roofs in London, July, 1952."

Friday, April 17th, 1953

Annual Dinner and Dance at the Parkway Hotel, Bramhope, Leeds, 7.0 p.m.

Wednesday, April 22nd, 1953

Annual General Meeting, followed by Mr. J. Dosson, M.I.C.E., A.M.I.Mech.E., (Member), on "Sea Defence Works."

All meetings to be held at The University, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17 The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg. 'Phone 34-1111 Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

THE MACLACHLAN LECTURE, 1952

The Design and Construction of a Three Bay Aluminium Aircraft Hangar at London Airport*

By L. E. Ward, A.M.I.Struct.E.

May, 1950, the Ministry of Civil Aviation invited ers for the design and construction of a three-bay gar at London Airport. With a view to reducing ntenance costs to a minimum, the roof structure to be fabricated in aluminium alloy but the stanch- at the discretion of the tenderer, could be either or aluminium. This paper will demonstrate that inium is a material to be seriously considered for of this nature.

pecification, Loading Conditions and Working Stresses

he original specification drawn up by the Ministry of Aviation allowed great latitude as far as the ctural design was concerned and it dealt mainly the functional aspect and standards of materials workmanship. The hangars were to be used for naintenance of aircraft and as an aid to the efficiency he operatives, a high degree both of thermal insula- and natural lighting was essential. As a means of cing the fire hazard, a deluge fire control system to be incorporated in the roof.

ach of the three bays of the hangar was to give a working area 125 ft. wide × 110 ft. deep, with a mum height of 30 ft. In addition, each was to be shed with a thermally insulated and power-operated giving a clear opening 125 ft. wide × 30 ft. high.

hat part of the specification concerning the intensity atural lighting was concise and simply stated that arrangement of glazing was to ensure a daylight or of 18 (computed according to T.P. No. 28 I.R. Building Research) over the majority of the area when the doors were in the closed position. roof, except where glazing occurred, was to be red with aluminium decking, possessing a thermal ation value of .35 B.Th.U./hr./°F./sq. ft.

order to guard against accidental damage, an high dado wall was to be constructed in brick- . This was surmounted by a 10 ft. deep stretch of cal glazing and the remaining wall was formed of .W.G. corrugated aluminium sheet. The area of heeting was to be lined with a material capable of standing the action of the deluge system, the high eratures likely to be encountered in the event of a and at the same time possessing a thermal insulation valent to that of the roof.

he loading conditions, which are severe when com- d with those in BSS449, take account of the exposed at London Airport and are as follows :—

Loading Conditions

e hangar was designed for the following specified ng conditions in addition to the dead loads :—

LOADING A

Superimposed Load (deemed to include snow) = 15 lb./sq. ft. on plan.

Superimposed Load (deemed to include lighting and heating services, etc.) = 2 lb./sq. ft. on plan.

LOADING B, C AND D

Wind Load. The unit wind pressure p = 25 lb./sq. ft.

LOADING B

An external wind pressure equal to p normal to the full area of vertical surfaces. In the case of main frames and bracing, this pressure was taken equally between the windward and leeward faces.

LOADING C

An external wind suction equal to p over half the length of the roof from the windward edge and $.5p$ over the remaining half, excepting that when the main doors are open to windward $.5p$ suction is taken over the whole roof.

LOADING D

An internal wind pressure of $.5p$ on all surfaces when the main doors are open to windward and an internal wind suction of $.5p$ when the doors are open to leeward or parallel with the wind direction.

Combination of Loading

Generally	(i)	A
Doors Closed	(ii)	A+B+C
,, ,,	(iii)	B+C
Doors Open	(iv)	A+B+C+D
,, ,,	(v)	B+C+D.

Each of the above conditions was to include thermal stresses due to a temperature range of ±45°F.

At the time of tendering the " Report on the Structural Use of Aluminium Alloys in Buildings " had not been published by this Institution. No other authoritative code being available, the working stresses and standards of construction were reconciled with BSS449 Use of Structural Steel in Building.

Struts were designed in accordance with the Perry Formulæ incorporating a value of $\eta = .0015 \frac{l}{r}$ and a load factor of 2. (This factor being reduced to 1.8 for loads due solely to wind.) In the case of members in tension and bending the safe working stress was taken as 13,500 lb./sq. in. representing a factor of safety of 3 on the ultimate tensile strength.

Rivets and Bolts

Bearing and shearing values of rivets and bolts were ascertained by testing sample specimens, and in each case

ven before the Institution of Structural Engineers at 11, Upper ve Street, London, S.W.1, on Thursday, November 13th, and awarded the MacLachlan Premium for 1952.

a factor of safety of 3 was used. This resulted in the following stresses :—

Permissible shear stress in NE5 rivets, 3.33 tons/sq. in.
" " " Black bolts, 4 tons/sq. in.
" bearing stress in HE10WP, 10.8 tons/sq. in.

In the case of the base castings the factors of safety were increased to 3.75 based on the ultimate stress.

A Brief Outline of Aluminium as a Structural Medium

Before proceeding with the details of this hangar, a brief resumé of the development of aluminium structures would not be out of place.

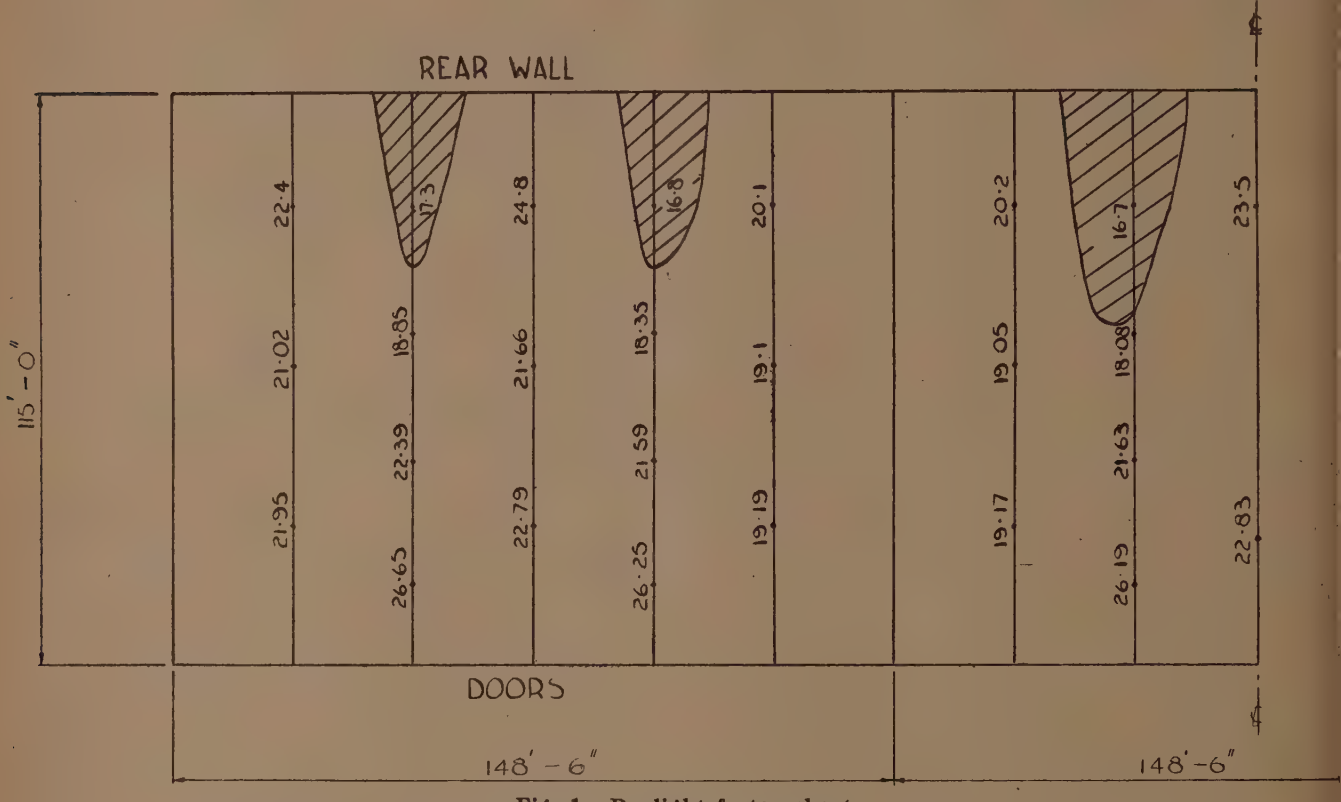


Fig. 1.—Daylight factor chart

N.B. Shaded areas are those which have daylight factors of under 18. Figures represent factors at particular points. Building is symmetrical about LC

As a structural material aluminium alloy has the following advantages :—

(a) A high resistance to corrosion, in consequence sections as thin as .08 in. can safely be incorporated in structures.

(b) Is easily extruded, thus permitting the designer to take advantage of this process in selecting special sections for a particular application.

(c) A high strength/weight ratio, thereby simplifying erection.

(d) The ease with which it can be worked eliminates the necessity of expensive equipment and results in low fabrication costs.

The principal disadvantages of aluminium are :—

(a) Its comparatively high cost.

(b) The relatively low modulus of elasticity of 10.3 × 10^6 lb./sq. in. may make deflection and not stress a criterion of design.

The most popular alloy for structural purposes is undoubtedly HE10WP, BS.1470, better known as AW10. It is available in the form of plate, rivets or extrusions. A higher strength alloy is HE15WP, but a copper content of 3.5—4.8 per cent. reduces its corrosion resistance. In their extruded and fully heat-treated conditions, these alloys have the following mechanic properties :—

	HE10WP	HE15WP
0.1% Proof Stress	15 ton/sq. in.	24 ton/sq. i
Ultimate Tensile Stress	18 ton/sq. in.	28 ton/sq. i
Elongation on 2 in.	10%	8%

HE10 owes its popularity to good corrosion resistanc ease and speed of extrusion and consequent lower pric Where minimum weight is of vital importance, HE is recommended.

To combat the high cost of aluminium, great ca should be taken to ensure that each square inch metal is used to its maximum load-carrying capacit

The small cost of an extrusion die encourages an engine to design his own sections, and one of the main facto involved is the nature of the load to which they will b subjected. In the case of tension members, the pe missible load is a function of the nett area, the shape the section having little effect.

Struts, however, present a different problem. Whe the slenderness ratio is large, it is usually more econ mical to spread the metal as far as possible in order t increase the radius of gyration and the permissible stres To this end, tubes, whether round or square, or battene channels are used. The tubes are more expensiv extrusions and battened channels more expensive i fabrication, but generally battened channels are mor practical for structures such as the one forming th subject of this paper. For short struts, when th permissible stress approaches the proof stress, angles ar usually the best solution. Apart from reducing th fabrication they are generally cheaper and consequentl on having a smaller perimeter, are stronger in torsion The latter is a point to be carefully considered in dealin with short struts. An effective method of improvin the efficiency of sections is to introduce bulbs or ribs a their extremities. This not only increases the torsiona strength and the radius of gyration, but protects th

relatively thin members against accidental blows, and in the case of angles, makes for easier extrusion. By virtue of its local stiffening, the bulb or rib allows thinner sections to be used.

Members in bending, as far as aluminium is concerned, should be avoided wherever possible. In the case of light members such as purlins, it is often impractical to use lattice members and indeed for relatively small spans, say up to fifteen feet, a simple beam section is the best solution.

Rivets up to $\frac{5}{8}$ in. dia. can be closed cold with the aid of a 25-ton squeeze riveting machine. It is of course desirable to use greater than $\frac{5}{8}$ in. dia. and indeed rivets up to $\frac{7}{8}$ in. dia. have been closed cold with a 25-ton machine. This has been accomplished at the expense of machining the end of the rivet shank and thereby reducing the closing load. A further method of approach, though still in the experimental stage, is to machine the snap in such a manner that the tendency is to spread the head and so assist in closing.

Connections may also be made by welding but in the heat-treated alloys this is accompanied by a loss of strength due to annealing. The original strength can be restored by reheat treatment but this is impractical in the case of large girders and consequently large structures are connected by either rivets or bolts.

Design

In view of the urgent need for this hangar, three weeks were allowed for the design and preparation of the tender. Consequently, the general details had to be decided quickly in order to obtain quotations from specialist firms for the roof decking, glazing and lining and arrangements demanding lengthy stress analysis were promptly ruled out. After considering various schemes it was decided to use two pinned portal frames spanning parallel to the door as principals, thus making each bay structurally independent. With this arrangement the depth of the portal frame leg provided sufficient width to house the doors in the open position. Each bay was structurally independent.

Attention was then given to glazing and it was found that the 10 ft. stretch of glazing on the lower roof slopes together with the 17 ft. wide deck light and the 10 ft. deep wall glazing, gave the requisite daylight factor over most of the floor, excepting that part immediately adjacent the doors. By the addition of the smaller deck light at the front of the building, the factor was increased to the required amount. Fig. 1 shows the contour.

The remainder of the roof is covered with aluminium decking. In principle, this consists of an 18 SWG corrugated sheet, the corrugations being spaced 6 in. apart and $2\frac{3}{4}$ in. deep. A sheet of insulation board spanning between the corrugations provides a base for the bituminous felt covering which, together with the insulation board, is bonded with hot bitumen. The decking is clipped to the purlin, clips being on alternate sides to stabilise the purlin flange. The weight of the decking is 4 lb./sq. ft. and it possesses a thermal insulation (U) value of .33 BTU's/hr./°F./sq. ft.

Fig. 2.—Interior view of the completed first bay

Building board manufacturers were contacted in an effort to obtain a material which would comply with the lining specification. No single board possessed all the required properties and so it was decided to use a $\frac{1}{4}$ in. thick cementitious asbestos board in conjunction with corrugated aluminium foil insulation. The foil was first stapled to the lining, the whole being panelled in aluminium sections, which were clipped to the sheeting rails.

The finished appearance of the interior of the first bay to be completed is shown in Fig. 2.

One bay is closed by sliding and folding doors, thermal insulation being provided by a layer of glass silk. Doors to the remaining bays consist of a series of L-shaped leaves, which slide and nest within each other. The base of the L forms the leaf, whilst the stem is a lattice girder transmitting the wind load to floor and roof.

In this door aluminium foil provides insulation. Both types of doors are electrically operated and can be opened or closed in one minute.

Standard aluminium rain-water goods could not be obtained in sizes sufficiently large to cope with requirements and were therefore specially fabricated. The valley gutters were formed from 10 SWG aluminium sheet and the main downpipes fabricated from sheet and channel section riveted to form a hollow rectangular section.

Having decided on the cladding, attention was diverted to the structure. A preliminary design revealed that the total dead load of the roof should be between 8 and 8½ lb./sq. ft., and the roof load resolved into a downward load of 15 lb./sq. ft., or an upward load of 25 lb./sq. ft. Taking the worst conditions for dead load the main frames were designed for a downward load of 25½ lb./sq. ft., or alternatively an upward load of 17 lb./sq. ft. A reversal of 72 per cent. A problem confronting the designer was to stabilise the inner boom of the main frame and to keep the weight and deflection of the purlins to a reasonable minimum.

weather tightness of the normal glazing and deck lights were employed.

In order to determine the loads with moderate accuracy a strain energy analysis was made. A Force diagram was drawn assuming no horizontal reaction at the base and a second diagram drawn for a horizontal reaction H. The loads in the members for both conditions were then tabulated and the value of the horizontal reaction found by means of the first theorem of Castigliano. This was then substituted in the table and the actual loads found. This table was also utilised to find the loads in the members due to temperature changes in the following manner.

Assuming no restriction to horizontal movement, the base of the portal frame would move outwards, due to vertical loads, a distance of 117.2 inches. The movement due to temperature variation of 45°F. on 146 ft 6 in., centre of pins, would be .933 inches. It follows that the horizontal load required to prevent movement would be $\frac{.933}{117.2} \times 100 = 0.8$ per cent. of the horizontal

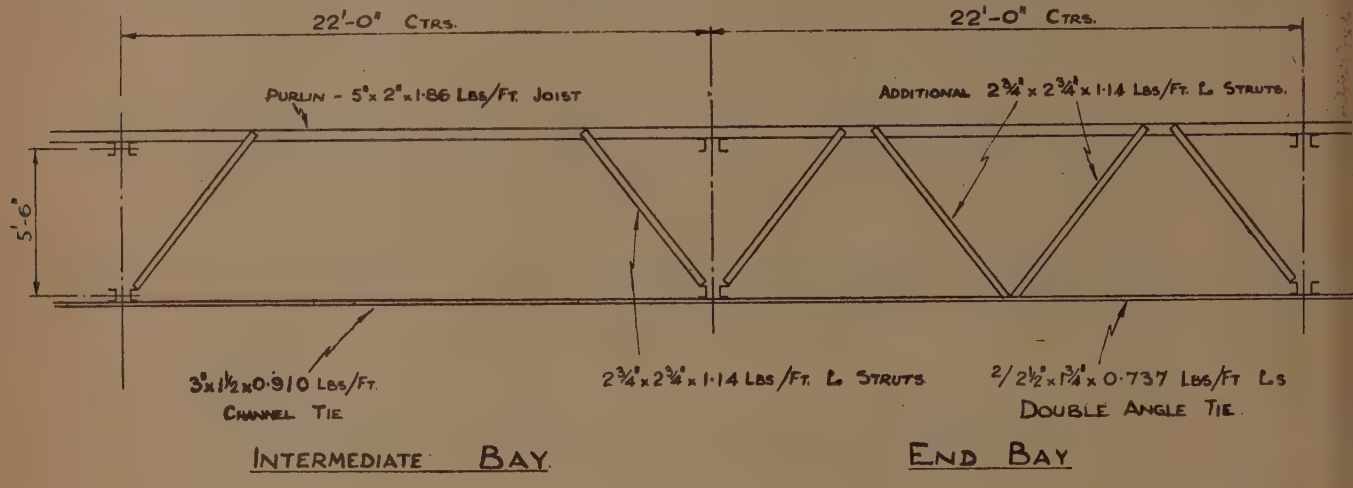

22'-0" Ctrs.
22'-0" Ctrs.
Purlin - 5"x 2"x1·86 Lbs/Ft. Joist
Additional 2¾"x 2¾"x 1·14 Lbs/Ft. L Struts.
5'-6"
3"x1½"x0·910 Lbs/Ft. Channel Tie
2¾"x 2¾"x 1·14 Lbs/Ft. L Struts
2/2½"x 1¾"x 0·737 Lbs/Ft L's Double Angle Tie
Intermediate Bay
End Bay

Fig. 3

It was decided to employ six frames equally spaced at 22 feet centres within the depth of the building. First thoughts were to design the purlins as latticed girders, but this scheme was rejected in favour of supporting the purlins at quarter points by struts springing from the inner boom of the main frame. A longitudinal tie was incorporated to transmit out of balance loads to the end bays, which, being fully latticed were capable of absorbing these loads. This system is shown in Fig. 3.

This arrangement of the purlins proved extremely economical. The main member is a 5 in. × 2 in. × 1.86 lb./ft. joist section and the struts which form an inverted Vee are 2¾ in. × 2¾ in. × 1.14 lb./ft. bulb angles. The joist is stressed to a bending stress of 10,000 lb./sq. in. and the calculated deflection of the purlin under full snow load is .5 in. It is worth noting that had a simple beam section, similar to those specified in BS1161 aluminium sections been used, the weight would have been of the order of 1.25 lb./sq. ft. compared with ½ lb./sq. ft. of the present design.

The actual span of the portal frame was dictated largely by the doors which in their parked position occupy a distance of 12 ft. 6 in. at each side of the clear opening. A nominal clearance of 6 in. between frames accounts for the dimension of 149 ft. centres of outer legs. The extreme 20 feet of each roof slope was pitched at 19° to facilitate the incorporation of normal patent glazing, the remainder of the roof being at 11° and 3.5° pitch, it was of course not possible to guarantee the

reaction due to vertical loads and consequently the additional load in a member would be 0.8 per cent. of the load induced by the horizontal reaction at the base.

As a further example strain energy was used to determine the vertical deflection of 6.4 inches at the apex and horizontal deflection of 2.4 in. at the eaves.

The forces in the portal frame having been found the members were designed. Double battened channels and bulb joists were used for the booms and heavily loaded internal members, the lesser loaded internals being of single channel section.

The gusset was cast in a ductile alloy LM6 in the form of a hollow box and is designed to transmit the entire load, in bending to the central pin. Steel bushes were incorporated to distribute the load and increase the bearing capacity. The base casting, which is not subjected to such high stresses, is made from LM4. This detail is shown in Fig. 4.

The rear gable framing is similar, in principle, to the roof. Latticed gable posts 5 feet deep spaced at an average centres of 25 feet, transmit the wind load to floor and roof. Sheeting rails are stayed and tied to the inner leg of the gable posts in a similar manner to the purlins and diagonal bracing in the plane of the sheeting is introduced to prevent undue sag. The rails were spaced at 4 feet centres to line up with the standard 8 feet lengths of lining board. The lattice gable posts can be clearly seen in Fig. 5, which shows the third bay in course of erection.

A specially designed joint, capable of transmitting orizontal but sliding under the action of vertical loads, introduced to prevent the gable posts being subject to ertical loads.

At the front of the building the top of the doors is otected by a canopy extending 10 feet from the timate frame, and hung from the ultimate frame by ub gable posts. The top of the canopy is covered with rrugated aluminium sheet and flat sheet fixed to orizontal framing members forms the soffit to receive e rubber sealing strip on the top of the doors. The availability of a suitable section made it necessary to over the remaining area provides additional dead weight.

The horizontal thrust on the base is transmitted by the wall of the casting bearing on the concrete. The only function of the bolts is to hold the building down, clearance holes in the casting ensure that the bolts are subjected only to tensile loads. With a view to avoiding duplication in the transmission of loads and also unwieldy connections, the side bracing was connected directly to the foundations by screwed rods. Provision was made for adjustment by enveloping the rods in a steel tube, the tube being firmly cast into the concrete.

Fig. 4.—Close up of hinge-base of portal

ilise steel channels for the top door guides which were pported from the projection of the main horizontal ind girder introduced to transmit the wind load on the oors to the side bracing. Ancillary bracing in the ane of the top boom caters for the wind on the upper art of the canopy.

The rafters of the rear portals are strengthened to rm the booms of wind bracing, which also incorporates e purlins, and transmits the wind loads to the rear side acing. Gussets vary in thickness, two $\frac{1}{4}$ in. being sed for the portal frames and main structure, two S.W.G. (.128 ins.) being used for the gable posts.

In the light of little experience of the expansion of uminium structures of this magnitude the problem as solved by designing each bay as an independent ructure. An expansion joint is also provided in the eeting and glazing in the centre of the side and gable alls in addition to a joint in the roof decking.

The foundations are constructed of normal reinforced ncrete and the unusual feature is that they were esigned to hold the hangar down as opposed to holding up. The larger type of foundation is designed to ater for an uplift of 75 tons and consists of a three-foot ab of reinforced concrete 23 feet long × 14 feet wide, e underside of which is 5 feet below floor level. A ur-foot square core of reinforced concrete projects om the centre of the slab to floor level and contains a ell to accommodate the base casting. Mass concrete The rod, the bottom of which had previously been greased, was twisted during the period the concrete was setting to permit movement in direction. A turnbuckle connecting rod to bracing permitted of adjustments in length.

Fabrication and Erection

The process of fabrication of an aluminium structure naturally follows on similar lines to its steel counterpart. The main difference results from the light weight of the components and the ease with which aluminium can be worked. Cranes do not form an essential part of the equipment and drills and saws are much smaller than those associated with steel fabrication.

The structure of this hangar was to be inspected by the A.I.D., and before fabrication commenced, a meeting was arranged between representatives of this body and the contractors, to agree standards of workmanship. As a result, samples of all stock material, whether bolts, rivets, sheet or extrusion, had to be mechanically tested to ensure that it was up to the specification. In many cases war-time procedure was reverted to in tracing new material back to its original cast in addition to testing batch samples. As the main site connections were to be made with turned and fitted bolts, a high degree of accuracy was necessary. The normal inspection staff worked in conjunction with the A.I.D. to ensure that this accuracy was maintained.

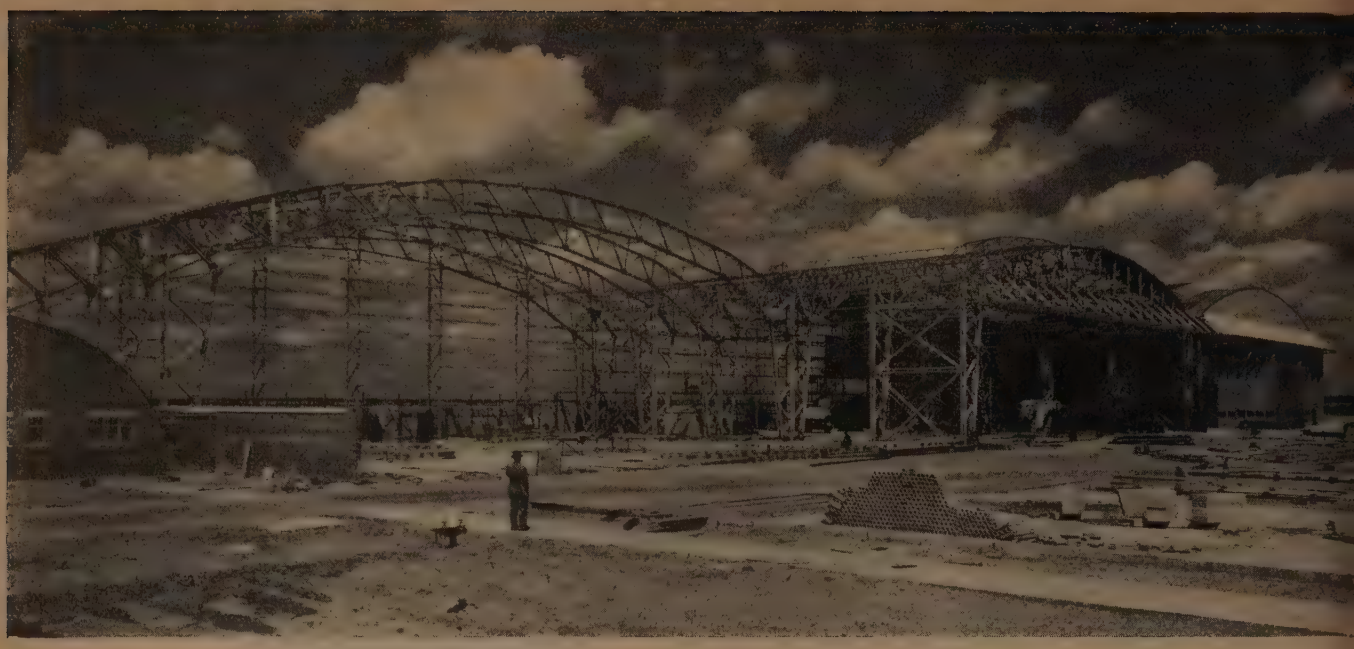


Fig. 5

The first stage in the fabrication was to set out the structure on the lofting floor, and the templates were made from the actual sections to be used, these being finally incorporated in the structure. The incoming material was cut to length on high-speed saws, following which pilot holes, generally $\frac{1}{8}$ inch less in diameter than the finished hole, were drilled. Batten plates were pre-punched and riveted to the main members in standard jigs, following which the gusset plates were riveted to the main booms and finally the internal members riveted on.

All rivets were cold driven closed, a 25-ton pneumatic squeeze riveting machine being used for $\frac{5}{8}$ rivets and a similar machine of 9-ton capacity used for $\frac{3}{8}$-inch diameter rivets. Although the smaller of these machines only weighs 28 lb., both were operated from a light aluminium framed trolley, balance springs being incorporated to give easier control of the machine. In the case of rivets not accessible, a long stroke percussion riveter was used to drive these by hand. Clearance of all holes was 1/64 inch over the rivet shank diameter and after driving the upset shank completely filled the hole. The portal frames were made in as long lengths as possible, the sizes being limited to those which could be conveyed to site on a 60-foot low loader. Each complete frame was temporarily shop-assembled and holes for fitted bolts reamed to ensure a good fit on erection. The completed sections were manœuvred with the aid of an available four-ton electric overhead travelling crane although, if necessary, they could have been man-handled.

Fig. 6.—Erection of half-portals

Fig. 7

Joints in the gutters were joggled prior to forming into trough section on a press brake. Outlets and stop ends ere bolted on and welded to ensure that the joints were atertight. All inner surfaces of the rectangular down-pes were painted with bituminous paint, prior to sembling with a zinc chromate jointing compound and veting up whilst in the wet condition. The same mpound was used for wet assembling the steel bolts, to revent electrolytic action between steel and aluminium, id the bolts finally painted over with a synthetic uminium paint. Precautions were also taken to revent corrosion of aluminium in contact with uncured ment by painting with bituminous paint.

Upon arrival at site the heavier components were f-loaded with the aid of a 3-ton 30-ft. jib crane. Two alves of adjacent portal frames were assembled side-by-le, the feet of the frames being connected to the nged bases and the apices resting on the concrete oor slab. After the purlins, ties, etc., had been fixed, rope was secured to connect the eaves to a three-ton winch situated about 75 yards outside the bases. A load applied by the winch then raised the portal frames, pivoted about their base pins, into position. After final adjustment had been obtained by jacking between the base casting and the foundation, the apex bolts were inserted and the base casting grouted in. The remaining components were then hoisted from the erected frames and fixed in position. Tubular scaffolding was erected alongside the rear and end walls and movable towers gave access to the roof.

The wall sheeting was drilled in position and hook bolted to the sheeting rails. A special staple was used for securing the aluminium foil to the lining board, which was then hoisted and fixed to the inner flange of the rail. The possible danger of glare on an unpainted sheet prompted the decision to treat the wall lining by the alchrom process. This dipping process is primarily to form a paint key, but it also served to improve the appearance of sheets. Fig. 6 shows the portal frames being raised into position, and in Fig. 7 the framework

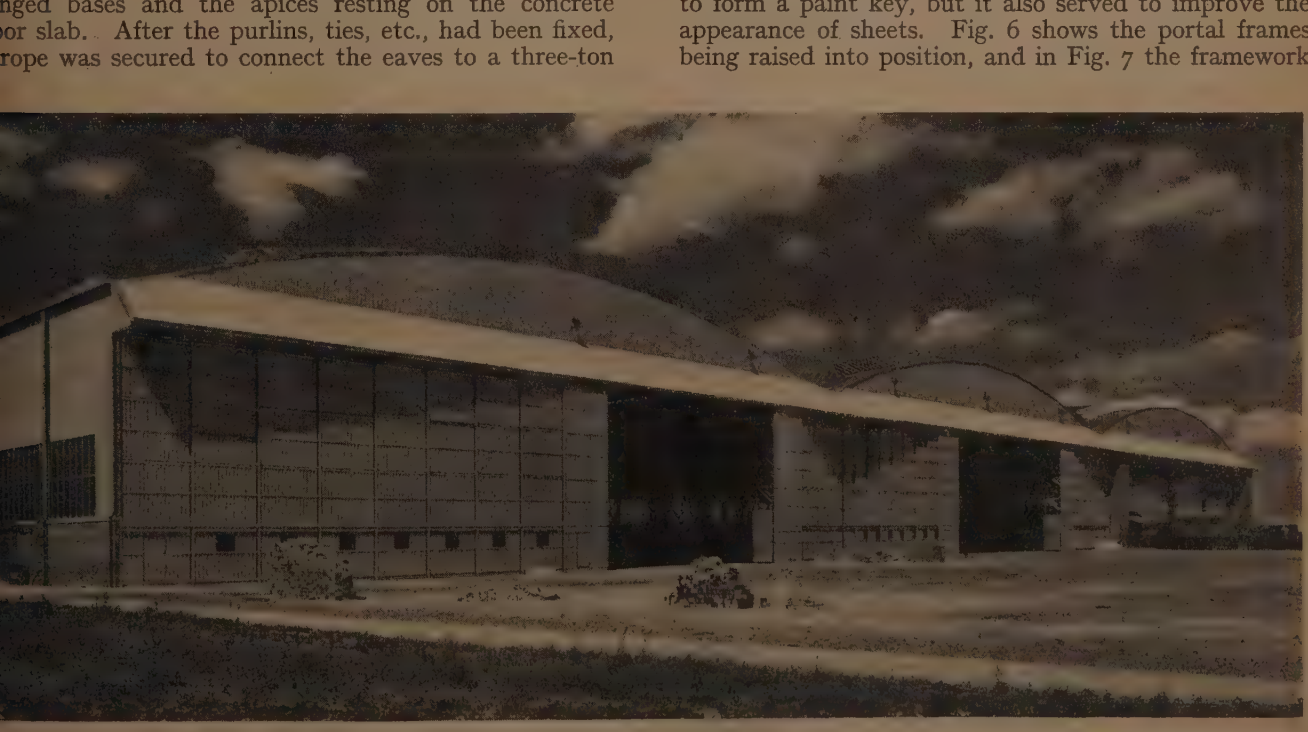


Fig. 8

to the 3rd bay is almost complete. The doors to the 2nd and 3rd bays are shown in Fig. 9.

Conclusion

The hangar, which despite a very wet winter, was completed within the period of a year, is an outstanding example of the use of aluminium in buildings. Apart from the structure, aluminium has been used in the roof decking, wall sheeting and glazing, in addition to forming the insulation for the walls. The doors, had they been constructed in any other material, would have been unwieldy. As will be seen by Fig. 8, the general shape enhanced by the various shades of green in the felt, glass and alochromed sheeting present a pleasing appearance. The efficiency of the glazing can best be judged on a winter's afternoon, when artificial lights are switched on an hour or two later than those in adjacent hangars. It may be said that the foundations are large, but when one considers the bearing pressure on the ground, ½ ton sq. ft., it is obvious that these hangars could be erected on the poorest soil.

This hangar is the forerunner of several which are, or are due to be erected, in various parts of the country and it seems that the future of aluminium aircraft hangars is assured.

Acknowledgements

The author wishes to extend his thanks to the Ministry of Civil Aviation and to Messrs. Structural and Mechanical Development Engineers, the main contractors for kind permission to publish this paper.

J. Bak, Esq., B.Sc., M.I.Struct.E., was Consulting Engineer for the design of the foundations and the sub-contractors for various parts of the cladding were Messrs. William Briggs—Roof Decking ; Messrs. Eastwoods—Insulated Linings ; Messrs. Williams and Williams—Aluminium Glazing ; Messrs. Wimpeys—Foundation Contractors.

Fig. 9

The Analysis of Multi-Bay Gabled Rigid Frames

By Arthur Bolton, M.Sc.Tech. (Graduate)

Summary

In presenting a previous paper[1], the author demonstrated the solution of a five-bay gabled frame of which each bay was unsymmetrical in geometry and loading. The solution was not then published, and in the present paper the calculation is set out in a form which can be applied to gabled frameworks with any inclination of the two rafters and with any loading. If the loading and the bays are symmetrical there is a considerable simplification of the general case.

Explanation of the method of calculation used and a fuller discussion of the constraints necessary to maintain equilibrium are to be found in the original paper.

Introduction

The analysis of a frame such as that shown in Fig. 1 is a tedious procedure. On the other hand, the analysis of the frame shown in Fig. 2 is very much simpler and can be completed in a few minutes.

[1]A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames. THE STRUCTURAL ENGINEER. Vol. XXX, 1952. p. 1.

The main cause of this difference is the fact that in the rst structure each bay is free to spread, and no obvious elationship can be found between the sways of the everal stanchions. This means that instead of being

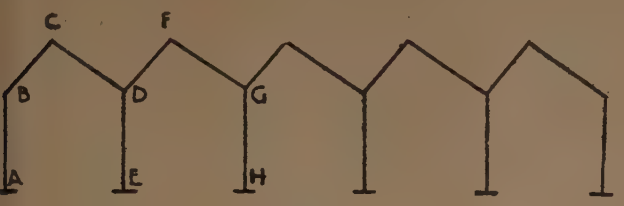


Fig. 1

ble to use one simple sway for the whole structure it ecomes necessary to consider the separate sway of ach stanchion.

A structure intermediate between these two types shown in Fig. 3. In this case the structure is similar

Fig. 2

o that in Fig. 2, but the beam is able to extend axially nd thus again there is the need to consider the separate ways of each stanchion. One method of solving this roblem is to consider the out-of-balance moment and he out-of-balance horizontal force at *B*, *D*, *G*. . . .

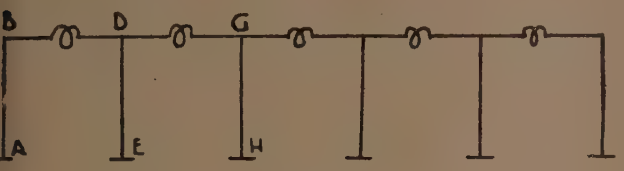


Fig. 3

By balancing these constraints in turn by rotations and ways the problem can be solved.

The structure of Fig. 1 can be converted to this dealised type by considering *BC* and *CD* as one member. For this composite member *BCD*, the stiffness and the arry-over factor to the opposite end for rotation of or *D* can be obtained. Moreover, the force required o extend the distance between *B* and *D* by a given mount can be calculated. Therefore, if a horizontal orce is applied at joint *D*, say, in Fig. 1, the force equired at *B* and *G* to prevent movement can be found. n other words, the structure of Fig. 1 may be considered as that of Fig. 3, in which the axial stiffness of the beams, the rotational stiffnesses and the carry-over factors are known. When the equivalent structure of Fig. 3 has been solved, the known forces and moments are applied to the real structure of Fig. 1 and the moments in the members calculated.

Constraints Due to Loading

If this method of solving the problem is adopted there exists the problem of transferring the fixed-end constraints on the real structure to the equivalent structure.

It was shown[2] that the equilibrium of *BCD* (Fig. 4) can be maintained against any loading by moments at *B*, *C* and *D* and horizontal and vertical force constraints at *B* and *D*. The moment at *C* will be dealt with later in the operations table, and the vertical forces are provided by the stanchions. The horizontal constraints at *B* and *D* due to any system of loading on span *BCD* remain to be calculated.

These constraints may be obtained by considering *B*, *C* and *D* as independently fixed. In this case there would appear, for a general load case, horizontal and vertical constraints at *B*, *C* and *D*. In fact *C* is not independently fixed, but is supported by the constraints at *B* and *D*. Therefore the forces which would have appeared at *C* are carried back to *B* and *D*, through the members *BC* and *CD*. The horizontal constraints at *B* and *D*, which are equivalent to the force constraints, which would have been required at *C*, have the following values :—

Horizontal Constraints at *B* and *D*

Due to H_C :—

$$H_B = -\frac{S_1}{L} \times H_C \qquad H_D = -\frac{S_2}{L} \times H_C$$

Due to V_C :—

$$H_B = \frac{S_1 S_2}{hL} \times V_C \qquad H_D = -\frac{S_1 S_2}{hL} \times V_C$$

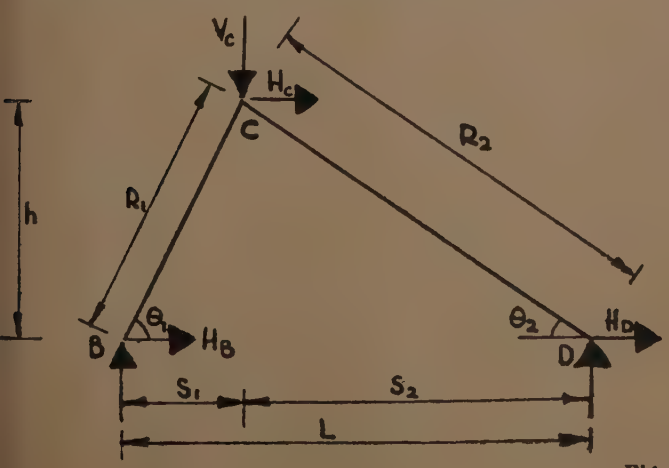
Calculation of Operations

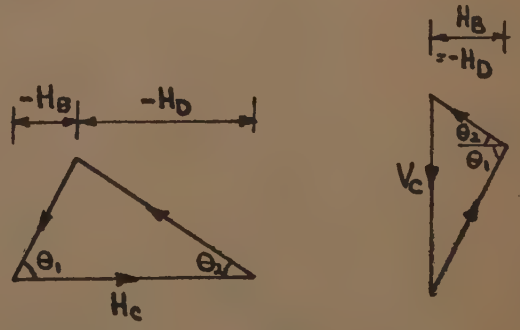
A. Moment Patterns

If one end of a prismatic member is rotated, the moment produced at that end is proportional to the stiffness $\left(\frac{I}{L}\right)$ of the member, and the moment produced at the far end is a half of that amount.

If one end of such a member is displaced by sway an amount δ relative to the tangent at the other end, the

[2]Example 6 p. 11. loc. cit.





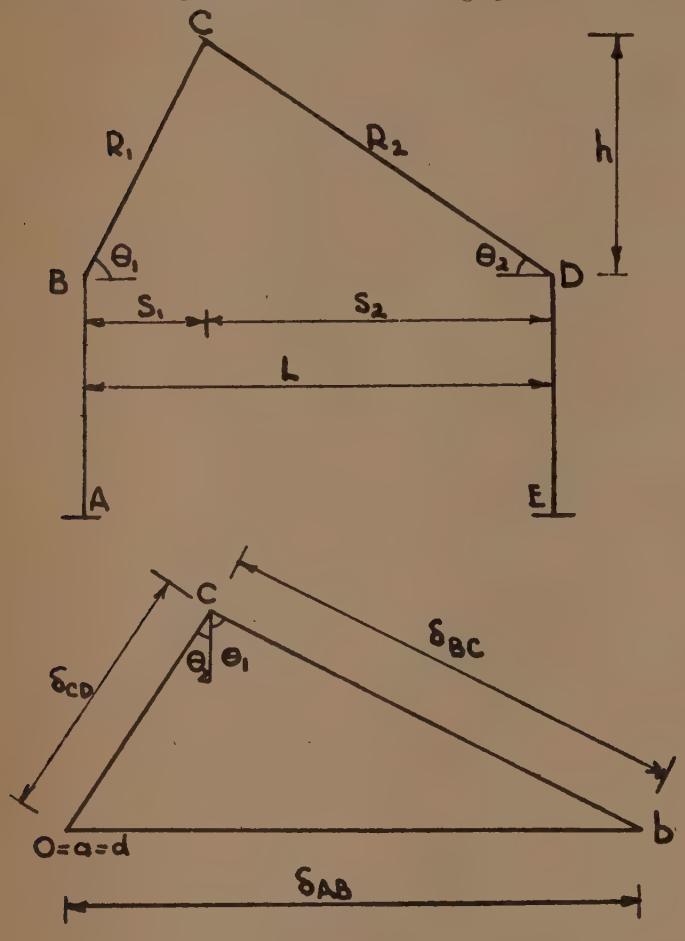
TRIANGLES OF FORCES AT C FOR H_C AND V_C.

Fig. 4

moment caused at each end will be proportional to $\frac{I\delta}{L^2}$.

For sways, therefore, as well as the $\frac{I}{L^2}$ value of each member, the deflection δ compared with that of each other member taking part in the particular sway is required.

These relative deflections may be obtained from a deflection diagram such as that of Fig. 5. This is the



WILLIOT DIAGRAM FOR DEFLECTIONS

Fig. 5

diagram for bay $ABCDE$ when A and D are fixed, and B is displaced an amount δAB to the right. From Fig. 5 : —

$$\frac{ab}{\sin(\theta_1+\theta_2)} = \frac{bc}{\cos\theta_2} = \frac{ca}{\cos\theta_1}$$

$$\therefore \delta_{BC} = \frac{\cos\theta_2}{\sin(\theta_1+\theta_2)} \times \delta_{AB} = \frac{R_1 S_2}{hL} \times \delta_{AB}$$

$$\delta_{CD} = \frac{\cos\theta_1}{\sin(\theta_1+\theta_2)} \times \delta_{AB} = \frac{R_2 S_1}{hL} \times \delta_{AB}$$

Exactly the same relative deflections are obtained if D is displaced and B held in position. Hence, knowing the $\frac{I}{L^2}$ value and the relative deflection for each member, the pattern of moments for any desired sway can be built up.

B. Force Patterns

There remains the problem of calculating the horizontal constraints at B and D for any pattern of moments in the bay $ABCDE$. The force at B may be considered as being made up of two parts, from the moments on stanchion AB, and from the forces required to maintain equilibrium of BCD.

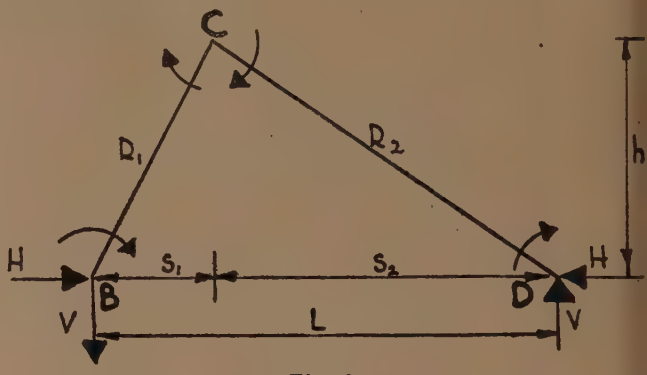


Fig. 6

From Fig. 6 the value of H due to moments in BC and CD may be obtained as follows :—

$$V = \frac{M_{BC}+M_{CB}+M_{CD}+M_{DC}}{L} \text{ (Moments about } B)$$

and

$$M_{CB} = -M_{BC}+Hh+VS_1$$

$$\therefore H = \frac{M_{BC}+M_{CB}}{h}\left(1-\frac{S_1}{L}\right) - \frac{S_1(M_{CD}+M_{DC})}{hL}$$

$$= \frac{S_2(M_{BC}+M_{CB})}{hL} - \frac{S_1(M_{CD}+M_{DC})}{hL}$$

$$\therefore H_B = \frac{S_2(M_{BC}+M_{CB})}{hL} - \frac{S_1(M_{CD}+M_{DC})}{hL} - \frac{M_{AB}+M_{BA}}{L_{AB}} \quad \ldots \quad (A)$$

Similarly

$$H_D = -\frac{S_2(M_{BC}+M_{CB})}{hL} + \frac{S_1(M_{CD}+M_{DC})}{hL} - \frac{M_{DE}+M_{ED}}{L_{DE}} \quad \ldots \quad (B)$$

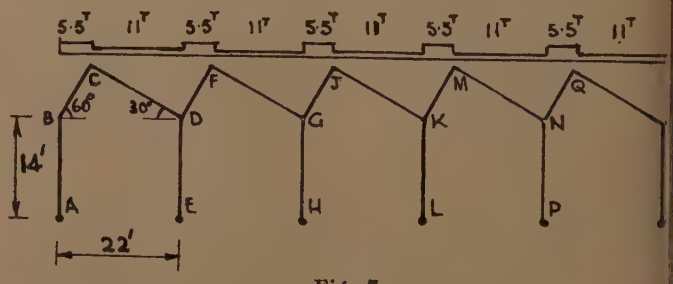


Fig. 7

Example

All the information necessary to construct an operations table is now available. As an illustration of the method, an actual example shown in Fig. 7 has been calculated. Since all the bays are identical it is con-

nient to consider the properties of two adjacent bays shown in Fig. 8. From a consideration of these two ys only, the properties of the eaves joints B and R, and so any intermediate joint, D, G, K or N, can be obtained

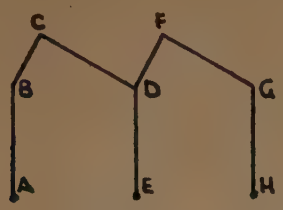


Fig. 8

The object of the operations table is to build up the omplex operations indicated in Fig. 9. Once these ave been obtained any constraints at B, D, G, K, N, R n the equivalent structure can be relaxed.

Calculation of Operations

For each bay the relative lengths of the members are : $_1=1$, $R_2=\sqrt{3}$, $S_1=\frac{1}{2}$, $S_2=\sqrt{3}/2$, $h=\sqrt{3}/2$, $L=2$.

Member	$K = \frac{I}{L}$	δ = Relative deflection for sway of B or D	$\frac{K}{L} \times \delta$
AB, DE	$\frac{1}{14} = .0715$	1	.00511
BC	$\frac{1}{11} = .0909$	$\frac{1 \times \sqrt{3}/2}{\sqrt{3}/2 \times 2} = \frac{\sqrt{3}}{2}$	.00716
CD	$\frac{1}{11\sqrt{3}} = .0525$	$\frac{\sqrt{3} \times 1/2}{\sqrt{3}/2 \times 2} = \frac{1}{2}$	.00138

Distribution Coefficients for Rotations

t B (A pinned) $BA = \dfrac{\frac{3}{4} \times .0715}{\frac{3}{4} \times .0715 + .0909} = .37$

$BC = .63$

t C $CB = \dfrac{.0909}{.0909 + .0525} = .635$

$CD = .365$

t D $DC = \dfrac{.0525}{.0525 + .0909 + \frac{3}{4} \times .0715} = .266$

$DF = \dfrac{.0909}{.0525 + .0909 + \frac{3}{4} \times .0715} = .462$

$DE = .272$

Distribution Coefficients for Sways

way of B (A pinned) $M_{BA} \propto \frac{1}{2} \times .00511$ i.e., say, 1.00

$M_{CB} = M_{BC} \propto .00716$ i.e., say, 2.82

$M_{DC} = M_{CD} \propto .00138$ i.e., say, .541

This information enables the first eight operations f Table 1 to be written down. The horizontal forces ssociated with each operation are calculated using quations (A) and (B) above.

The eight basic operations must be combined to build p the required complex operations of Fig. 9. To btain the operations for H_B and M_B operations (1), 2) and (6) only need to be considered since the only llowable displacements are the rotation of B and C nd the sway of B.

In operation (1) there is an out-of-balance moment at C which can be removed using —.315 parts of operation (2) to give operation (9). In operation (2) there is an out-of-balance moment which can be removed using —.318 parts of (1) to give operation (10). It is easy to balance operation (6) using (9) and (10) since there is no interaction. Operation (12) shows the result of this balancing and the value of H_B is seen to be .3709. By multiplying operation (12) by $\frac{1}{.3709}$ operation (13) is obtained in which H_B is unity. From this operation the first pattern of Fig. 9 can be obtained.

Moreover, operation (9) is seen to be the operation required for the second pattern, M_B of Fig. 9, except that there is a force of .0288 units as H_B. By subtracting .0288 of operation (13) from (9) and multiplying by $\frac{1}{.900}$ the required operation is obtained.

In a similar way the patterns for the remaining complex operations of Fig. 9 are obtained in Table 1.

When these patterns have been obtained it is time to consider the particular loads the structure is carrying.

Fixed-end Constraints for the Equivalent Structure

$$-M_{BC} = M_{CB} = \frac{5 \cdot 5^2}{12} = 2.52 \text{ tons/ft.}$$

$$-M_{CD} = M_{DC} = \frac{11 \times 16.5}{12} = 15.15 \text{ tons/ft.}$$

Thus the dead load produces an out-of-balance moment of 12.63 tons/ft. at each ridge point. This moment is balanced using operation (2) of Table 1 and the effect on the constraints at B, D, . . . noted.

In addition a vertical force would have been required at each ridge point, had these been independently fixed, of value :—

$$V_C = \frac{5.5}{2} + \frac{11}{2} = 8.25 \text{ tons.}$$

Using the expression previously obtained,

$$H_B = -H_D = V_C \times \frac{S_1 S_2}{hL}$$

$$= 8.25 \times \frac{\sqrt{3}}{4}$$

$$= 3.57 \text{ tons.}$$

Relaxation of Constraints

Table 2 shows the method of relaxation used. The constraints due to the F.E.M.s at B, D, . . . R are entered, and the force constraints due to the vertical force at the ridge points are added. The effect on the constraints of using Operation 2 to balance the ridge points is also noted, and the sum of these three sets of values gives the total constraint to be relaxed.

The next step is to guess the amount of horizontal force and moment which will be needed at each joint B, D, G, K, N, R. It is here that experience of other similar problems is valuable, for time will be saved in the relaxation if a close estimate is obtained. It will be observed that, because of the tendency of every bay to

Table 1

	OPERATION	MOMENTS											FORCES		
		BA	BC	CB	CD	DC	DE	DF	FD	FG	GF	GH	H_B	H_D	H_G
1	ROTATION OF B	·37	·63	·315									·0479	-·0743	
2	" " C		·318	·635	·365	·182							·0606	-·0606	
3	" " D				·133	·266	·272	·462	·231				·0105	·0456	-·0545
4	" " E							·318	·635	·365	·182			·0606	-·0606
5	" " G									·247	·495	·505		-·0195	·0165
6	SWAY B→	-1·00	2·82	2 82	-·544	-·544							·544	-·471	
7	" D→		-2 82	-2·82	·544	·544	-1·00	2·82	2·82	-·344	-·544		-·471	1·015	-·471
8	" G→							-2·82	-2·82	544	·544	-1·00		-·471	·544
	OPERATIONS H_B AND M_B WITH D FIXED - OPERATIONS 1, 2 AND 6 ONLY														
9	(1) - ·315 ×(2)	·37	·530	·115	-·115	-·057							·0288	-·0552	
10	(2) - ·318 ×(1)	-·118	·118	·535	·365	·182							·0454	-·0370	
11	2·025×(9)+2·527×(10)	·449	1·371	1·585	·691	·346							·1731	-·2052	
12	(6) - (11)	-1·449	1·449	1·235	-1·235	-·890							·3709	-·2658	
13	(12) × 1/·3709	-3·91	3·91	3·33	-3·33	-2·40							1·00	-·720	
14	(9) - ·0288 × (13)	·483	·417	·019	-·019	·012							0	-·0344	
15	(14) × 1/·900	·537	·463	·0211	-·0211	·0133							0	-·03825	
	OPERATIONS H_D AND M_D WITH B AND G FIXED - OPERATIONS 2,3,4,7.														
16	-·133×(2)+(3) - ·231× (4)		-·0423	-·0845	·0845	·2417	·272	·3885	·0845	-·0845	-·0423		-·01856	·03966	-·0405
17	2·276[(2)-(4)] +(7)		-2·097	-1·374	1·374	·958	-1·00	2·097	1·374	-1·374	-·958		-·3337	·7407	-·3337
18	(17) - $\frac{2·088}{·9027}$ × (16)		-2·001	-1·182	1·182	·4072	-1·619	1·214	1·182	-1·182	-·861		-·2814	·6504	-·2416
19	(18) × 1/·6504		-3·077	-1·816	1·816	·626	-2·488	1·865	1·816	-1·816	-1·324		-·448	1·000	-·372
20	(16) - ·03966 × (19)		·080	-·0125	·0125	·2169	·371	·3145	·0125	-·0125	·010		-·00076	0	-·0258
21	(20) × 1/·9024		089	-·0139	·0139	·241	·411	·348	·0139	-·0139	·011		-·0008	0	-·0286
	OPERATIONS H_G AND M_G WITH D FIXED - OPERATIONS 4, 5, 8.														
22	(4) - ·182×(5)							·318	·635	·320	·092	-·092		·0641	-·0576
23	-·247×(4) +(5)							-·0786	-·1572	1572	·450	·505		-·0345	-0015
24	(8)+2·38×(22)+·478×(23)							-2·102	-1·380	1·380	·978	-·978		-·335	·4062
25	(24) × 1/·4062							-5·18	-3·41	3·41	2·41	-2·41		-·825	1·00
26	(22) + ·0015 ×(25)							-·0863	-·1623	·1623	·4536	·5014		-0357	0
27	(26) × 1/·955							-·0903	-·170	·170	·475	·525		-·0373	0

spread, a large overcorrection of the constraints will be required. The trial solution used in Table 2 is probably better than average because the solution of a frame, geometrically similar but with different stiffnesses, was available.

When a trial solution has been decided the effects on the constraints, both at the point on which the operation is being carried out and by carry-over to the adjacent joints, are noted. For example, a force constraint of —10.0 was guessed at B. From pattern (a) of Fig. 9 the carry-over is seen to be

$$-10.0 \times -.72 = +7.2 \text{ to } H_D$$
$$-10.0 \times -2.4 = +24.0 \text{ to } M_D.$$

After such terms have been entered the constraints are totalled. If the estimate had been the exact solution, there would be no constraint remaining. In general, however, there will be residuals, but far smaller than the original fixed-end constraints.

These residuals are then attacked using the patterns of Fig. 9. For example, after the trial solution +1.52 tons remain as constraint at H_D. This is released by using —1.50 parts of pattern (b). By repeated operations on the residuals in turn, they are brought down to a level at which they can be ignored.

At this stage the amount of each complex operation of Fig. 9, which has been used in the solution, is obtained by adding the corrections to the trial solution.

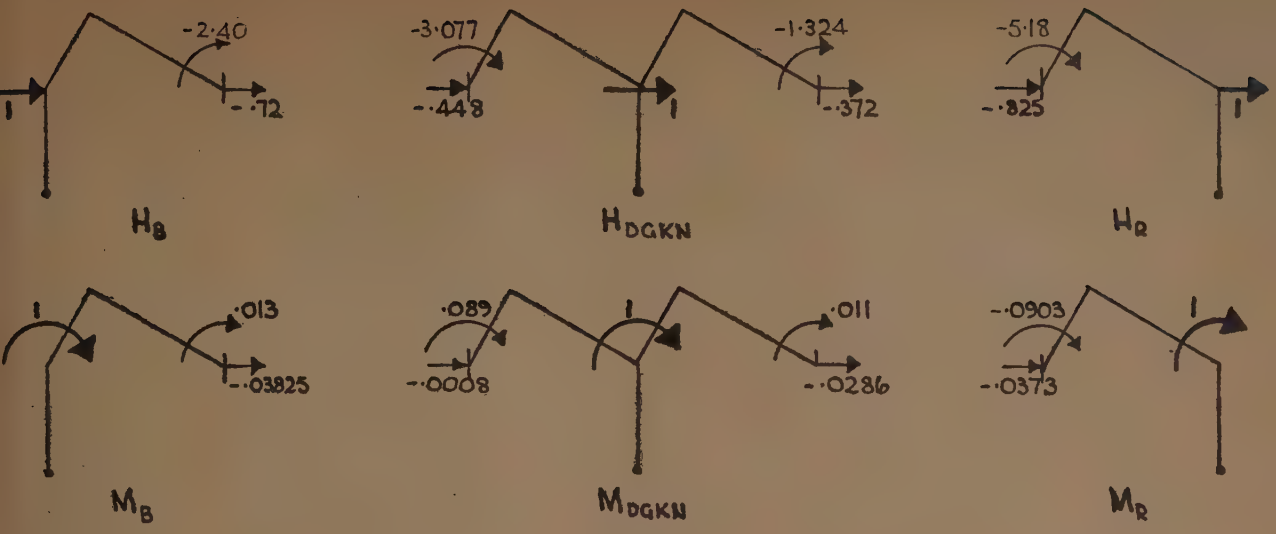


Fig. 9

Relaxation Table 2

	B		D		G		K		N		R	
	H	M	H	M	H	M	H	M	H	M	H	M
TRIAL SOLUTION	-10·0	-30·0	-10·0	-60·0	-7·0	-40·0	-3·0	-20·0	+2·0	0	+5·0	-15·0
CORRECTIONS	+·50	-·80	-1·50	-·20	-·30	-·50		-2·40	-·10	+2·40	+·10	
	+·05		+·25		-·04			-·20	-·01		+·01	
			-·05									
TOTAL	-9·45	-30·8	-11·30	-60·2	-7·34	-40·5	-3·0	-22·6	+1·89	+2·40	+5·11	-15·0
ORIGINAL F.E.M.'s		-2·52		12·63		12·63		12·63		12·63		15·15
	3·57										-3·57	
(2) TO BALANCE C,F,J,M,Q.	·77	4·02		6·32		6·32		6·32		6·32	-·77	2·30
TOTAL TO BE RELAXED	4·34	1·50		18·95		18·95		18·95		18·95	-4·34	17·45
	-10·00		7·20	24·0								
		-30·0	1·15	-·39								
	4·48	30·77	-10·0		3·72	19·24						
	·05	-6·34		-60·0	1·72	-·66						
			3·14	21·54	-7·00		2·60	9·27				
			·03	-3·56		-40·0	1·14	-·44				
					1·34	9·23	-3·0		1·12	3·97		
					·02	-1·78		-20·0	·57	-·22		
							-·90	-6·15	+2·0		-·74	-2·65
									-4·12	-25·9	+5·0	
									·56	1·35		-15·0
RESIDUALS OF TRIAL SOL	-1·13	-3·07	1·52	·54	-·20	-1·02	-·16	1·63	·13	-1·85	-·08	-·20
	·67	4·61	-1·50		·59	1·99			-·08	-·52	·10	
	·50		-·36	-1·20			·04	·31	-·10		·04	·13
	-·11	-·77	·25		-·09	-·33	-·02	·21		2·40	-·07	·03
					·02	-·21		-2·40	·07	-·03		
			·13	·92	-·30		·11	·40				
		-·80	·03	-·01								
	-·07	-·03	·07	·25	·02	·43	-·03	·16	·02	—	-·01	-·04
	·05		-·04	-·12				·03	-·01			·01
			·02	·12	-·04		·01	·05				
	·02	·15	-·05		·02	·06			-·01	-·05	·01	
			—	-·04		-·50	·01	-·01				
		-·01	·	-·20	-·01	·01		-·20	-·01			
RESIDUALS UNCORRECTED	—	·11	—	·01	-·01	-·02	-·01	·03	-·01	-·05	—	-·03

MOMENTS IN MEMBERS IN TONS FT.

M_{BA} = — 9.45 × —3.91 —30.8 × .537 = +20.4
M_{DE} = —11.30 × —2.49 —60.2 × .411 = + 3.43
M_{GH} = — 7.34 × —2.49 —40.5 × .411 = + 1.65
M_{KL} = — 3.00 × —2.49 —22.6 × .411 = — 1.82
M_{NP} = + 1.89 × —2.49 + 2.4 × .411 = — 3.73
M_{RS} = + 5.11 × —2.41 —15.0 × .525 = —20.17

M_{BC} = —2.52+(12.63 × .318) — 9.45 × 3.91 —30.8 × .463 —11.30× —3.08 —60.2 × .089 = —20.33
M_{DF} = 1.50 —11.30 × 1.865 —60.2 × .348 —7.34 × —3.08 —40.5 × .089 = —21.58
M_{GJ} = 1.50 — 7.34 × 1.865 —40.5 × .348 —3.0 × —3.08 —22.6 × .089 = —19.09
M_{KM} = 1.50 — 3.00 × 1.865 —22.6 × .348 +1.89 × —3.08 + 2.1 × .089 = —17.56
M_{NQ} = 1.50 + 1.89 × 1.865 + 2.4 × .348 +5.11 × —5.18 —15.0×—.0903= —19.29

M_{CB} = 2.52+(12.63×.635) — 9.45×+3.33 —30.8 × .021 —11.30 × —1.816 —60.2 × —.0139 = — .27
M_{FD} = 10.54 —11.30 × 1.816 —60.2 × .0139 —7.34 × —1.816 —40.5 × —.0139 = +3.06
M_{JG} = 10.54 — 7.34 × 1.816 —40.5 × .0139 —3.0 × —1.816 —22.6 × —.0139 = +2.43
M_{MK} = 10.54 — 3.00 × 1.816 —22.6 × .0139 +1.89 × —1.816 +2.4 × —.0139 = +1.33
M_{QN} = 10.54 + 1.89 × 1.816 + 2.4 × .0139 +5.11 × —3.41 —15.0 × —.170 = — .90

M_{DC} = 15.15+(12.63×.182) — 9.45 × —2.40 —30.8 × .0133 —11.30 × .626 —60.2 × .241 = +18.17
M_{GF} = 17.45 —11.30 × —1.324 —60.2 × .011 — 7.34 × .626 —40.5 × .241 = +17.44
M_{KJ} = 17.45 — 7.34 × —1.324 —40.5 × .011 — 3.0 × .626 —22.6 × .241 = +19.39
M_{NM} = 17.45 — 3.00 × —1.324 —22.6 × .011 + 1.89 × .626 + 2.4 × .241 = +22.98
M_{RQ} = 17.45 + 1.89 × —1.324 + 2.4 × .011 + 5.11 × 2.41 —15.0 × .475 = +20.17

M_{CD} = —15.15+(12.63×.365) — 9.45 × —3.33 —30.8 × —.021 —11.30 × 1.816 —60.2 × .0139 = +.27
M_{FG} = —10.54 —11.30 × —1.816 —60.2 × —.0139 — 7.34 × 1.816 —40.5 × .0139 = —3.06
M_{JK} = —10.54 — 7.34 × —1.816 —40.5 × —.0139 — 3.0 × 1.816 —22.6 × .0139 = —2.43
M_{MN} = —10.54 — 3.00 × —1.816 —22.6 × —.0139 + 1.89 × 1.816 + 2.4 × .0139 = —1.33
M_{QR} = —10.54 + 1.89 × —1.816 + 2.4 × —.0139 + 5.11 × 3.41 —15.0 × .170 = +.90

Knowing the solution for the equivalent structure, the solution to the real structure can be obtained. The internal moments corresponding to the patterns of Fig. 9 are contained in operations (13), (15), (19), (21), (25), and (27) of Table 1. The moments caused by balancing the ridge points with Operation 2 are also known. When these are added to the fixed-end moments the solution is complete. The final computation may seem laborious but it will be seen that blocks of calculation can be obtained at one setting of the slide rule.

Alternative Load Cases

If it is desired to obtain the effect of some other load case on the same structure, say, for instance, a wind load, only a small part of the calculation needs to be repeated. All that is needed is to relax the new constraints with the same patterns of Fig. 9 and obtain the internal moments.

Appendix

Non-Prismatic Members

Since the stanchions of such a frame as that under consideration often carry gantries and have an abrupt change of section at gantry level, there is the need to calculate the stiffness and carry-over factors for a non-prismatic member. As a simple illustration the member *ABC* of Fig. 10 is here analysed.

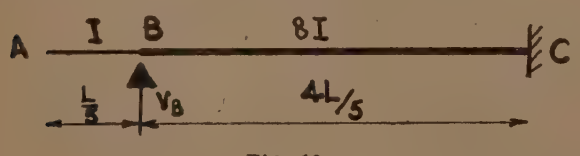


Fig. 10

Fig. 10 shows a member *ABC* fixed at *C* and with a change of section at *B*. It is required to find the stiffness of *A* and the carry-over to *C* when *A* is rotated.

The problem is more easily visualised when taken i stages. If *B* is fixed and *A* rotated there will be known moment caused at *B*, and a known force *V* will be required to prevent vertical displacement of *B*

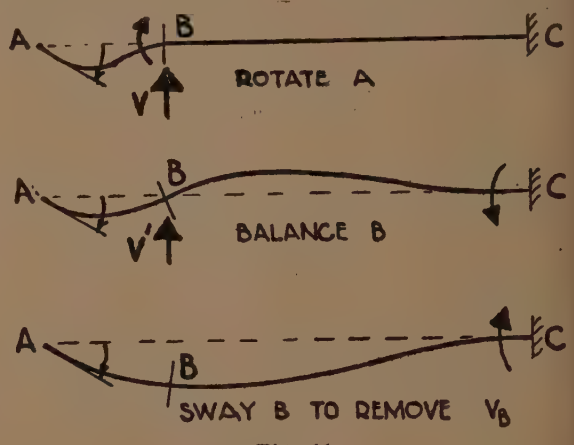


Fig. 11

If *B* is now allowed to rotate, the out-of-balance momer at *B* can be removed, but some force constraint *V* will remain. If *B* is next allowed to sway the force *V* can be removed, although some smaller out-of-balan moment will be reintroduced at *B*. By repeating the operations of rotation and sway of *B* the solution will l obtained. Alternatively, the amount of rotation ar sway of *B*, which accompanies any rotation of *A* m be obtained directly without successive correctio Fig. 11 shows the basic operations. The shear equatic can be obtained from Fig. 10.

$$V_B = \frac{M_{AB}+M_{BA}}{L/5} - \frac{M_{BC}+M_{CB}}{4L/5}$$

$$\therefore V_B \times L = 5\left(M_{AB}+M_{BA} - \frac{M_{BC}+M_{CB}}{4}\right)$$

TABLE 3

	Operation	A		B		C	$V_B \times L$
1	Rotate B	1	2	4	2		$15/2$
2	Sway B	2	2	—1	—1		$45/2$
3	(1) × 3 —(2)	1	4	13	7		0
4	(1) —(2) × 6	—11	—10	10	8		$-\frac{255}{2}$
5	Rotate A	34	17				255
6	(3) × —1	— 1	—4	—13	—7		0
7	(4) × 2	—22	—20	20	16		—255
8	(5) + (6) + (7)	11	—7	7	9		0
9	Sway A	51	51				510
10	(3) × —3	— 3	—12	—39	—21		0
11	(4) × 4	—44	—40	40	32		—510
12	(9) + (10) + (11)	4	— 1	1	11		0

The two basic operations for the rotation and sway of B are entered at the head of Table 3. For convenience it is better to obtain an operation (3) which will relax moment at B without affecting the vertical force, and another (4) which will relax vertical force at B without affecting the moment constraint. Using (3) and (4) there will be no interaction, and one cycle will be sufficient to remove the constraints at B.

Moreover, operations (3) and (4) are important on their own account since they give the fixed end moments at A and C for moments or forces applied at B.

For example, if a load of one ton is applied at B and L is measured in feet, it is seen from (4) that :—

$$\text{F.E.M. at } A = -\frac{11 \times 2 \times L}{255} = -\frac{22L}{255} \text{ tons ft.}$$

$$\text{F.E.M. at } C = \frac{16L}{255} \text{ tons ft.}$$

Likewise a moment of 1 ton ft. applied at B will result in fixed-end moments given by (3) as follows :—

$$\text{F.E.M. at } A = \frac{1}{17} \text{ tons ft.}$$

$$\text{F.E.M. at } C = \frac{7}{17} \text{ tons ft.}$$

Thus any fixed-end constraints at top and bottom of stanchions due to eccentric gantry loads or surges can be found.

To obtain the stiffness of A and the carry-over to C the pattern for the rotation of A is written down (5). A rotation of 34 units is made in the particular case because it is known that operation (3) will relax 17 units at B). Enough of operations (3) and (4) to balance the moment and vertical force at B are now added and the sum, operation (8), gives the pattern of moments for rotation of A without B being in any way restrained.

From this pattern the carry-over factor from A to C is seen to be $\frac{9}{11}$ since 11 units of bending moment applied at A result in 9 units appearing at C.

The stiffness of the composite member ABC is obtained by comparing it with the stiffness of AB which has a

$$k \text{ value of } \frac{I}{\frac{L}{5}} = 5\frac{I}{L}.$$

AB subjected to a certain rotation at A exerts a moment of 34 units. ABC subjected to the same rotation at A exerts a moment of 11 units. Therefore the relative stiffness of ABC is :—

$$\frac{11}{34} \times \text{relative stiffness of } AB$$

$$= \frac{11}{34} \times 5\frac{I}{L} = 1.62\frac{I}{L}$$

To analyse a structure containing such a member there remains the problem of the size of the moments produced by a sway of A relative to C. This problem is solved by allowing A to sway relative to B and using (3) and (4) to balance the moment and vertical force caused at B by this operation.

Operation 12 shows the final pattern of moments. Again the sway moments are compared with those of AB. The $\frac{K}{L}$ value for AB is $\frac{25I}{L^2}$ and the original sway produced end moments of 51 units. The same sway on the composite member ABC produces 4 units of bending moment at A. Therefore the same moment is produced by the sway of A as though it were one end of a uniform member with

$$\frac{K}{L} = \frac{4}{51} \times \frac{25I}{L^2}$$

$$= 1.96\frac{I}{L^2}$$

Similarly, the moment produced at B by sway is the same as though it were one end of a prismatic member with

$$\frac{K}{L} = \frac{11}{51} \times \frac{25I}{L^2}$$

$$= 5.39\frac{I}{L}$$

Hence, the pattern of moments for any sway involving member ABC can be obtained.

If loads are applied along the stanchion it will be necessary to find the fixed-end moments for the composite member. To do this the appropriate fixed-end constraints for the individual spans AB and BC are entered as in line (5) and relaxed using operations (3) and (4).

This paper was written in the Department of Building and Structural Engineering, College of Technology, Manchester ; Head of Department, W. B. McKay, M.Sc.Tech., M.I.Struct.E.

Introduction to the Vacuum Concrete Processes*

Discussion on Monsieur I. Leviant's Paper

In the Chair, Mr. J. E. SWINDLEHURST, O.B.E., M.I.C.E., M.I.Struct.E., President of the British Section, Société des Ingénieurs Civils de France, supported by Mr. E. GRANTER, B.Sc.(Eng.), M.I.C.E., President of the Institution of Structural Engineers.

The CHAIRMAN, in introducing M. Leviant, said it was a great honour to have a man of his calibre to address them, as a brief outline of his career would show. M. Leviant was educated at l'Ecole Polytechnique de Paris and l'Ecole Nationale des Ponts et Chaussées, and had commenced his administrative career in the Corps des Ponts et Chaussées as Engineer-in-Charge of the Control of Hydro-Electric projects in the south-west of France. Later he was engaged on similar problems on behalf of an important private company, in the course of which he prospected the Fouta-Djallon in French Guinea, formulating the plan for the requisite hydro-electric equipment.

In the year 1944, M. Leviant was sent on an official mission to the United States, where he acted as technical adviser to his President, M. Jean Monnet, on the French Supply Council, later succeeding him in that office in the years 1946 and 1947.

In the course of his assignment in America he met Mr. Billner, the inventor of the vacuum concrete process, and returning to private life M. Leviant founded in Paris, in association with Mr. Billner and a financial group, the Société du Vacuum Concrete, which in the last four years had been entrusted with the development of these new processes in many lands.

He was also the author of a work entitled "Hydro-Electric Schemes," and many other technical publications. In short, he was a man of great eminence, with a master's knowledge and grasp of his subject.

Mr. LEVIANT then gave his address, which was illustrated by numerous slides and a film.

The CHAIRMAN said were it not for the fact that it was their custom for the Chairman to propose the vote of thanks to the lecturer, there would undoubtedly be a great rush for that honour as there could hardly be one person in the room who would not like to have that opportunity. They had listened with rapt attention to a lecture which made history. The vacuum concrete process was one which, although it was well outside its experimental stage, opened the door to far greater things. M. Leviant had been speaking from knowledge and from practice, and if they had learned one thing it was that they could not afford to neglect any system or any type of operation which would effect that economy which was so necessary in these days if the capital works which they so much desired to carry through were to be made practicable.

Earlier in the evening M. Leviant had mentioned to him that he was not finding Britain quite so conservative in its methods as he had anticipated—speaking, of course, not in the political sense! He had been very pleased to hear that, but there was still much to be done.

Finally, he proposed formally that a most hearty vote of thanks be accorded to M. Leviant for his interesting, entertaining and inspiring lecture.

Mr. E. GRANTER, seconding the vote of thanks, said our enjoyment of the address had been added to considerably by M. Leviant's excellent command of the English language. They were all glad when such eminent Frenchmen, who were scientists and engineers, came over and gave details of Continental practice and progress. In some ways they were ahead of this country, and the knowledge of the interesting work being undertaken on the Continent kept us alert.

During the previous spring he had driven across France and had seen a great deal of interesting engineering and building work : villages which had been entirely demolished by the Germans had been practically rebuilt with the most attractive houses and public buildings.

As President of the Institution of Structural Engineers, he warmly welcomed the British Section of the Société des Ingénieurs Civils de France, and also their Chairman, Mr. Swindlehurst, who was a Past-President of the Institution.

Discussion

Dr. D. F. ORCHARD said that the vacuum concrete process had been used quite extensively abroad, notably in the United States of America and in France, but he felt that even in those countries its full potentialities had not yet been realised and that if it was to progress further, both abroad and in this country, it might be necessary to revise the normally accepted methods and standards of design.

Any new subject did not really start to make its full progress until people began to discuss it and ask questions about it, and for that reason they were particularly fortunate in having with them the man who had pioneered the process in Europe.

In his paper M. Leviant had offered an explanation of the action which went on in the vacuum concrete process and had inferred that there was some other action beyond the mere extraction of water and compaction of the concrete, and support was lent to that possibility by some tests which had been carried out in the research laboratories of the Cement and Concrete Association. In the course of those tests it had been noticed that a concrete of 1 to 6.2 mix which had been processed down to a water-cement ratio still as "high" as 0.59 was quite capable of standing up on its own on a 12 in. face after stripping of the shuttering, and he felt quite certain it would have stood up on a considerably higher face. It was significant that this concrete, when dug out and worked up again, resumed the characteristics that would normally be associated with such a mix at that water-cement ratio, and it would be very nice to have M. Leviant's further observations on that point.

The speaker had emphasised that the process was one of squeezing the water out and not one of sucking it out : in the initial stage of the process the rate of

*Read before a Joint Meeting of the Institution of Structural Engineers and the British Section of the Societe des Ingenieurs Civils de France at 11, Upper Belgrave Street. London, S.W.1, on Thursday, November 27th, 1952. Published in THE STRUCTURAL ENGINEER, Vol. XXX, No. 11, pp. 249-258. (Nov. 1952.)

npaction was high and therefore the rate of with-
wal of the water was high, and after a time—about
een minutes, normally—the rate of compaction and
refore of withdrawal of the water slowed up. That
ne behaviour took place irrespective of the water-
nent ratio at the start, and therefore the final water-
nent ratio was dependent on the initial water-cement
io.

In the initial stages the concrete was compacted until
friction between the aggregate particles became so
at that further compaction could not take place,
e compaction progressed from the processing mat
o the thickness of the concrete until such time as the
istance to the further withdrawal of the water through
already compacted concrete near the mat became
great, and the inference to be drawn from that, in
opinion, was that if something could be done to
luce the internal friction between the particles, the
ep part of the relation could be continued further.
vo ways occurred to him in which that could be done :
stly, by simultaneous vibration and, secondly, by
addition of some chemical to the mixing water to
luce the friction between the particles, and he would
glad of further information on those two possibilities.

The rate of reduction of the water-cement ratio in
initial stages was dependent on such variables as
thickness of the concrete being processed, the initial
ter-cement ratio, and the quality of the mix, but
er the initial period the rate of extraction did not
pear to be so sensitive to those variables.

The remarks which he had made referred to top
ocessing. As far as he knew up to now no similar
ts had been carried out in this country for side pro-
ssing, but if the behaviour was similar then, under
rmal circumstances, a reduction in the water-cement
io of between 0.15 and 0.2 could be expected. That
ing so, there was a possible way by which the normally
cepted mix design methods could be applied to the
cuum process. If the final water-cement ratio was fixed
give the strength required then from that could be
tained the necessary initial water-cement ratio to
ve the required final ratio, and that initial water-
nent ratio would enable the mix to be designed to
ve the required workability.

The fact that the water-cement ratio at the start was
gher, would enable the required workability to be
tained with a considerably leaner mix. Alternatively,
would be possible to start with a lower water-cement
io and end up with a very much lower one and there-
e a considerably stronger concrete, and he would
e M. Leviant to say in which direction he considered
maximum economy could be obtained : i.e., either
using leaner mixes or by going on, as they normally
uld, and obtaining a very much stronger concrete.
he considered the latter course was likely to be the
tter one then some revision of the Code of Practice
uld become necessary, and steels of higher tensile
lue than the normal mild steel bars might well have
be used.

Reverting to the rather more practical side, he felt
at the vacuum concrete process would open up great
ssibilities in the surface finishing of *in situ* work. The
rmal methods of brushing and scraping and hosing
ich had been applied up to now chiefly to pre-cast
rk might be applied in future to *in situ* work. A
ssible difficulty was that the type of mix which gave
best surface finish was not likely to be the best mix
the vacuum process, but it might well be possible to
ercome that difficulty.

M. Leviant, in his paper, had made the statement
at the bond between vacuum concrete and old concrete
was excellent, and it would be of interest to know whether he considered that it might afford a possible way of overcoming the difficulties of obtaining adequate bond between, say, a $\frac{3}{4}$ in. or 1 in. screed and an old concrete floor. The pressure exerted by the atmosphere, amounting to about three-quarters of a ton per square foot, might well help considerably to improve the bond with old work when very thin screeds were being laid, and further, the reduction in the water-cement ratio should assist by reducing the relative shrinkage. There had been a case in America where a 2 in. topping had been placed on an old concrete road : he had not so far seen anything in the American technical Press about the success or otherwise of that experiment, and he wondered whether the speaker had any information on it.

M. Leviant, in reply, said that with regard to the special state of the pseudo-solidified concrete, at the end of the treatment the channels filled with water inside the concrete remained under depression ; they remained at an almost zero pressure which was in equilibrium with the atmospheric pressure owing to the fact that each of those channels ended with very curved menisca : if the channels were of the order of 3 microns in diameter an equilibrium could exist. The fresh concrete was in a state which, to a certain extent, was similar to prestressing : the water channels were not under tension but were under depression, which was similar to the tension of the prestressing wire ; they pulled on menisca, the latter playing the part of anchorages, and thus held together the skeleton of the aggregates, and that, to a certain extent, "prestressed" structure explained the additional self-supporting capacity of the concrete.

As regards the initial withdrawal of water being more rapid than the withdrawal afterwards it should be made clear that the water-cement ratio during the treatment and as long as it has not been completed was an average and did not correspond to an evenly distributed water-cement ratio. In fact it was like having a cold wall and putting heat on one side of it : if the average temperature of the wall was measured one would see it growing, but that did not mean that the average temperature would be evenly distributed. The initial withdrawal of the water was higher becuase the layers which were in direct contact with the filter were the first to lose the water, and from there the flow of water went very rapidly because it did not have to cross an already compacted layer, but as the penetration of the treatment went deeper, so it had to cross thicker and thicker already compacted masses of concrete. It was the general diffusion law of physics which applied to heat transfer and to soil consolidation as well as to the vacuum treatment.

The more rapid penetration was, of course, a target in which every user of vacuum concrete was interested, because theoretically it could go on indefinitely, subject to the cement being completely inactive, which no one desired. In fact the penetration became very slow after one foot in thickness, but there were methods of accelerating it. The use of vibration during the treatment, which is in fact a usual practice, would accelerate it, and if the vibration was made by the filter itself the water was drawn by the vibrating body towards the area from whence it was more easily and more rapidly extracted. All plastifiers facilitated the treatment and speeded it up, and everything which facilitated the movement of particles was favourable.

As regards the choice to be made between a leaner mix (or there was another possibility—a more plastic mix to start with) and stronger concrete, the answer

was in fact that the variation of water-cement ratios through the treatment could be such that both advantages could be gained at the same time. In other words a more plastic and workable mix could be used, or a leaner mix, but nevertheless at the end a stronger concrete could be obtained. Naturally, the proportions could be adjusted to the local economic conditions. For example, in the United States they aimed at the maximum saving in labour, which meant that they favoured the most workable mixes and were not interested in a saving of the cement content nor in the increase of strength to the same extent as in Europe. In Europe the tendency was towards stronger final products, which meant that instead of starting with liquid mixes they used plastic mixes and used vibration during the treatment, and at the end a stronger concrete was obtained which better suited the economic circumstances.

The bond of a thin crust of concrete applied to an old concrete was in fact improved, for three reasons : firstly, the new concrete could be poured in a plastic state, which meant that it had more chance of coming into close contact with the old concrete ; secondly, the compaction which pressed it against the old concrete, and thirdly, the reduction of the shrinkage of the new concrete, due to the reduction of the water-cement ratio, which reduced the causes of " divorce " and of shrinkage cracks between the two. The road toppings which had been made in the United States several years previously had been tested and were still in excellent condition, and the bond was found to be very good. The same type of application had been repeated in a case of vertical elements : a dam which had been completely damaged through frost had a sheet of concrete cast against the old concrete, which was bonded again and resisted very well.

The treatment of very thin precast slabs of large size was one of the developments on which they were now working, and M. Leviant produced a sample cut from a concrete slab which had been cast in a size of 4 ft. × 8 ft. × ½ in. thick, being reinforced with chicken wire. Such concrete slabs reacted like steel plates ; one surface, the bottom, when cast was shiny, like marble, through a special device, and the resistance which they had obtained on tests made in Austria gave 1,000 lb./sq. in. tensile strength, measured in flexion.

Mr. J. Singleton-Green (Hon. Librarian), referring to the splendid talk given by M. Leviant, said that whilst on holiday in Paris he had spent three days in M. Leviant's office to learn about vacuum concrete. Each day they had started at nine o'clock in the morning and either M. Leviant or one of his assistants had talked to him until half-past six at night, the only break being for a marvellous lunch each day. At the end of the third day M. Leviant had said, " Well, there you are—you now have a brief outline of it " !

There were two questions he wished to ask. The first was quite a fair one, i.e., could the author assure them that what applied to ordinary cement applied equally to all the special cements, such as rapid-hardening Portland, high alumina, quick-setting Portland, etc. ? The answer might be rather important—he was including cements containing admixtures.

The second question, which was perhaps not quite a fair one, dealt with cost : they had had comparative figures, but could M. Leviant give them some idea of what the difference in overall cost would be, in any special case, having taken into account all the various factors ?

M. Leviant, in reply, said the question with regard to the type of cement was very pertinent because the vacuum treatment gave different results with different cements. They were in the process of making tests on different types of cement and had found that with certain of them peculiar things happened, but he hoped to be able in the near future to give additional information. For the time being all his statements applied to all Portland cements (including high early strength) and furnace slag cements ; before using other cements preliminary tests should be made. He had applied the vacuum treatment with various admixtures and it had worked very well—sometimes even more rapidly and more deeply.

The question of cost was a very complicated and difficult one, and if he had known that Mr. Singleton Green had that in his mind he would have kept him in Paris for three more days ! It should be clearly borne in mind that vacuum concrete was not a panacea for all ills ; on the contrary, it had a very definite field of application and outside that field it might be not only not advantageous but actually absurd. In fact the main problem was to define the field of its economic application, the main purpose of the process being savings—quality coming in addition to that. The main fields in which savings could be obtained were those of elements of large size (at least in one direction, like beams, or in two directions such as slabs, arches, pipes and walls). It did not apply to elements with no big dimension, like a brick, or to elements with three big dimensions, like a dam (except surface treatment). For elements of small size which could be produced by vibration alone vacuum concrete was not usually advisable because in that case it possessed no economic advantage : if one could vibrate very strongly and very thoroughly a small element using very dry concrete which immediately could be stripped that was the most that could be expected. Hence there was a field where vacuum concrete should be completely ruled out, and that was the policy which they had tried to apply.

In the field where advantages could be gained by the vacuum method the economic advantages were not the same in all cases. In vertical structures, for instance, precast or cast *in situ*, the savings were bigger than for horizontal structures, even though in horizontal structures such as floors cast *in situ* with the shutter beneath, one could strip the shutter twice as early, but it was not as striking as removing vertical shutters after fifteen minutes. To give a figure which was, of course, only a very rough indication, the savings in those cases where the process was recommended were in the neighbourhood of twenty to twenty-five per cent. of the value of the goods produced. For instance, on a large project where 5,000,000 sq. ft. of slabs were produced 250,000,000 francs had been saved on a work of 1,000,000,000 francs.

Mr. H. Kaylor (Associate-Member), referring to the case of the vacuum process being applied to columns or similar members where the mats might be on, say, three or even four surfaces, asked whether one was likely to obtain a difference of compaction so that in the interior of the column there might be a plastic area or zone with a denser area on the periphery which might give rise to cracks internally. Also, he would like to know what was the best amount of water to remove.

He had been very pleased to note the increase in bond strength and the reduction in shrinkage because one could make good use of that in prestressed concrete, particularly for pre-tensioned units. M. Leviant had mentioned in his paper that vacuum concrete had a

greater impermeability, and if one visualised the capillary channels as having an easy flow to the surface was the concrete going to be more impermeable or less ?

To try to improve the concrete before they started to put it in the job, had consideration been given to applying the vacuum process in the mixer ? Would it help to get a stronger concrete to start with ?

M. LEVIANT, in reply, said that with regard to the possibility of making a bad treatment, this, like every other process, could be rightly or wrongly applied, and if it was not well applied one could have bad results. If one stopped in the middle of the process then one had not finished it.

As regards the treatment of four sides of a column it was exactly the same as having a column which one wanted to warm up and asking whether, if one put four warm panels on the four sides the column would be completely warm and at the same temperature or whether there would remain a centre zone of cold ? It depended upon how long the column was heated, which meant that the matter of the length of treatment had to be considered. In addition, the association of vibration to the treatment permitted both saving in time and more uniform result at the end of the treatment. In vibrated (not vacuum treated concrete) one had at the end of the vibration sometimes an uneven distribution of water-cement ratio : in the case of shutter vibrators one had sometimes a water-cement ratio 15 per cent. higher in the neighbourhood of the shutters, but that was not the case in vacuum treatment : the outside would be the most dry but if it was properly done the inside would be almost identical.

It was not possible to say what was the best amount of water to be removed, except that they were trying to remove as much water as they could because they all had one target, namely, a water-cement ratio of 0.28 : that was what they lived for but never achieved. The world record in vacuum processed concrete was 0.31 and they had never been able to go below that, which meant that they need have no fear of going to 0.27, which would be bad, so his advice was to remove as much water as possible. The amount of water which was removed was in fact not the result of their desires but of the proper way of designing the grading of the aggregates, because the final stage which was reached was the result of the grading. Sieve analysis was one of the things on which they had made extensive research.

With regard to the question of " capillary channels," as a result of the percolation of water these were not formed during the treatment. It was not to be thought that cement particles, instead of being in a crowd, would suddenly—because of the flow during the treatment—stand in line and form sort of alleys. In point of fact the intersticial spaces remained in the same general form as before, which meant that they went in all directions like the streets in the old quarters of Paris : after the treatment they remained in that state but the general scale was reduced. They were squeezed together but they remained as tortuous as they were. The best proof was that the tunnel at Mucone, lined with vacuum concrete pipes, had now been put under pressure and on one mile there were losses of three-quarters of a litre per second. Vibration combined with vacuum was the essential factor for obtaining the maximum compaction and imperviousness.

Vacuum in the mixer was a very old idea and patent which had nothing to do with the present process. It had been a complete failure because it started with the idea that it would be a good thing to close the vacuum mixer completely and suck the air out. It had been expected that one practical result of such vacuum mixers was that they avoided air being entrapped, but that was not true : air was entrapped, except it was depressed air ; and so in fact they were useless.

The CHAIRMAN, speaking on behalf of the British Section of the Société des Ingénieurs Civils de France, thanked the Institution of Structural Engineers for all their assistance and the proceedings then terminated.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on February 26th, 1953, at 5.55 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership.

STUDENTS

CROSSLEY, Norman, of Manchester.
DE BONO, Joseph Alan, of Sliema, Malta, G.C.
ETTINGER, Norman Barry, of Ilford, Essex.
JARRETT, George Edward, of London.
KING, Gaius Stephen Hurworth, of Wellington, N.I, New Zealand.
MASTERMAN, Jack William Edwin, of London.
SWALE, John, of Otley, Yorks.
ZAFAR, Sheikh Mohammad, of Jhang Naghiana, Punjab, Pakistan.

GRADUATES

ALLEN, David, B.Sc.(Civil) Bristol, of London.
BLOOMFIELD, Ralph I'Anson, A.R.I.B.A., of London.
CHOWDHRY, Bashir Ahmad, B.Sc.(Civil) Lahore, D.I.C., of Gidea Park, Essex.
DAYA, Ronald Stephen, of Prestwich, Lancs.
GENOVESE, Joseph Lewis, of Birkirkara, Malta, G.C.
GLOVER, Frank, of Manchester.
GORECKI, Zenon, of Pendlebury, Lancs.
KOTWAL, Gangadhar Tryambakrao, B.E.(Civil) Poona, of Chandor, District Nasik, Bombay Province, India.
LETMAN, John Albert, of West Wickham, Kent.
MERCER, George, of Nottingham.
PATWARDHAN, Achyut Gangadhar, B.E.(Civil)Hons., Bombay, of Poona, India.
RANADE, Hemchandra Trimbak, B.E.(Civil) Bombay, of Poona, India.
RATHOD, Mahindra Valjibhai, B.E.(Civil) Bombay, of Bombay, India.
READY, Reginald Frederick, of London.
SMALLRIDGE, James Mervyn, of Chester.
SMITH, Geoffrey Ernest, B.Sc.(Eng.) London, A.C.G.I., of London.
VAUGHAN, William George, of Tipton, Staffs.
WEATHERS, David Anthony, of Shepperton, Middlesex.
WHITWORTH, Charles Victor, of Clay Cross, Derbyshire.

Members

Irani, Kekhashru Rustom, B.E.(Civil) Bombay, B.Sc. (Hons.) Edinburgh, of Bombay, India.
Rotzinger, Francis Canning, of Swansea, Glam.

Transfers

Students to Graduates

Campbell, Peter Leonard, of Coulsdon, Surrey.
Hotz, Peter Martin, of London.
Manning, Brian Hawthorne, of Prestwich, Lancs.
Pryke, John Fordham Speaight, B.A.(Hons.) Cantab., of Wormley, Herts.

Graduates to Associate-Members

Asher, Peter John, B.Sc.(Eng.) South Africa, of Durban, South Africa.
Barnas, Stanislaw Adam, of London.
Bletcher, Raymond Leonard, of London.
Chilton, William Gordon Eley, M.A.(Hons.) Cantab., of Wolverhampton, Staffs.
Halsall, Robert, B.Sc.(Civil) (Hons.) Glasgow, of Calcutta, India.
Henry, Robert Nelson, of Auckland, New Zealand.
Hicks, Edward George, of Ilford, Essex.
Mirvis, Lionel, B.Sc.(Eng.) Rand, of Johannesburg. South Africa.

Associate-Members to Members

Soper, Major Richard John, M.Eng. Sheffield, A.M.I.C.E., A.M.I.Mun.E., of Victoria, British Cameroons, West Africa.
Sung, Reginald Henry Lothian, B.Sc.(Eng.) Hong Kong, of Seria, State of Brunei, Borneo.

Members to Retired Members

Cook, Major Robert Ewing, O.B.E., of St. Helier, Jersey, C.I.
De Roemer, Major Charles William, J.P., M.I.E.E., of Farnham, Surrey.
Firth, Ernest Hargreaves, of Bramley, Leeds.
Jenkins, Archibald Augustus William, of Bournemouth.
Swarbrick, John, F.R.I.B.A., of London.
Wulff, Johan Christen, B.Sc., of Cobham, Surrey.

Re-Admissions

Associate-Members

Clark, Alan Douglas, of Johannesburg, South Africa.
De Souza, Angelo Maria, of Bardez Goa, Portuguese India.

OBITUARY

The Council regret to announce the death of Harry Ewart Holden, Scott Hamilton Hume (Members), Charles George Monro (Associate), William Girvan Brown and William John Taylor (Associate-Members).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of Gerald Patrick Barrett, Herbert James Cook, Charles Owen Hooper, Francis Ernest Wentworth-Sheilds (Members; William Fenton Anderson, Frederick Stanley Vyall (Retired Members); George Arthur Baldwin, Clifford Worthington (Associates); Angus George Forbes Alexander, Ernest Foulis Dempster (Associate Members); Kenneth Douglas Bundy, Dermont McMurchy Campbell, Leonard James Colyer, Angus Charles Etherington Duncan, Gerald Keats, Kenneth John Leighton, Keith Dudley Sandifer (Graduates); Desmond Vyvyan Spindler (Student).

WITHDRAWAL OF RESIGNATIONS

The Council have accepted the withdrawal of the resignations of the following :—

Peter John Tanner, of Peterborough (Student)
Joseph Gimpel, of Cobham, Surrey (Associate Member)

EXAMINATIONS

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 14th and 15th, 1953 (Graduateship), and July 16th and 17th, 1953 (Associate-Membership).

ANNUAL DINNER

The Annual Dinner will be held at the Dorchester Hotel, Park Lane, London, W., on Friday, May 1st, 1953, at 7 o'clock for 7.30 p.m. The Principal Guest will be the Rt. Hon. Harold Macmillan, M.P., Minister for Housing and Local Government. Speeches will conclude at approximately 9.30 p.m., and will be followed by dancing until 2.0 a.m. Particulars have been circulated to members.

SUMMER MEETING

The first Summer Meeting of the Institution to be held since 1939 will take place in Scotland from May 20th to 22nd, 1953. The Headquarters will be at the Royal Technical College, Glasgow, and members will have an opportunity of enjoying the hospitality of the Scottish Branch for certain items in the programme, particulars of which were circulated with the February issue of the Journal. On Thursday, May 21st, a paper entitled "Engineering Aspects of the New Atomic Research Extension, University of Glasgow" will be given by Mr. D. M. Hamilton, B.Sc., M.Eng., M.I.C.E. and Mr. J. Hardie Glover, A.R.I.B.A. Introduced by Professor W. T. Marshall, B.Sc.(Eng.), Ph.D., M.I.Struct.E., A.M.I.C.E. Members who wish to participate and who have not yet applied for the requisite application forms are urged to write to the Secretary without delay.

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :—

Council for Codes of Practice for Buildings
Mr. Stanley Vaughan (Vice-President).

Executive Committee of the Council for Codes of Practice
Mr. Walter C. Andrews, O.B.E. (Past-President).

London County Council District Surveyors' Examination Board
Mr. L. E. Kent (Hon. Secretary) and Mr. E. R. Knight (Member) (re-appointment).

FORTHCOMING MEETINGS

Wednesday, April 15th, 1953

By courtesy of the British Section of the Societé des Ingenieurs Civils de France, members of the Institution of Structural Engineers and their friends are invited to attend a meeting to be held at the Iron and Steel Institute, 4, Grosvenor Gardens, London, S.W.1, on Wednesday, April 15th, 1953, at 6 p.m., when Monsieur A. Lazard, Chef de la Division des Ouvrages d'art de la S.N.C.F., will give a paper entitled "La Plastification de l'Acier-Doux" (Flexural Plasticity in Mild Steel). Members who so desire are cordially invited to take part in the discussion.

The following meetings will be held at 11, Upper elgrave Street, London, S.W.1 :—

Thursday, April 16th, 1953

An additional Ordinary Meeting will be held at 1, Upper Belgrave Street, London, S.W.1, on Thursday, pril 16th, 1953, at 6 p.m., when a paper on " Economy n the Use of Structural Materials," will be given by Ir. G. A. Gardner, O.B.E., M.I.Struct.E., Chief Struc- ural Engineer, Ministry of Works, Mr. R. Morton, .Eng., M.I.C.E., Deputy Civil Engineer-in-Chief, dmiralty, and Mr. L. R. Creasy, B.Sc., M.I.C.E., eputy Chief Structural Engineer, Ministry of Works.

A limited number of advance copies of the paper will e available and may be obtained on application to the ecretary.

Thursday, April 23rd, 1953

An Ordinary General Meeting for the election of nembers will be held at 5 p.m.

Cancellation of Meeting

Owing to unforeseen circumstances, it has been ecessary to cancel the Ordinary Meeting which was to ave been held on Thursday, April 23rd, 1953.

Thursday, May 14th, 1953

An additional Ordinary Meeting will be held on hursday, May 14th, 1953, at 6 p.m., when Mr. G. M. rost (Member) will give a paper on " The Structural spect of Power Station Design in South Africa."

Members wishing to bring guests to the Ordinary leetings announced above are requested to apply to the ecretary for tickets of admission.

Thursday, May 28th, 1953

Ordinary General Meeting for the election of members, p.m.

Thursday, June 25th, 1953

Ordinary General Meeting for the election of members, p.m.

ANNUAL GENERAL MEETING

The Annual General Meeting of the Institution of tructural Engineers will be held at the Royal Technical ollege, George Street, Glasgow, on Thursday, May 21st, 953, at 5.30 p.m.

BENEVOLENT FUND

The Annual General Meeting of the Voting Con- ributors to the Institution of Structural Engineers' enevolent Fund will be held at the Royal Technical ollege, George Street, Glasgow, on Thursday, May 21st, 953, at 5.45 p.m.

PERSONAL

Mr. K. R. Hoskins (Associate-Member) has set up in ractice as a Consulting Structural Engineer at Karamia hambers, Crown Lane, Wollongong, New South Wales, ustralia.

Mr. J. F. Bagnall (Associate-Member) has taken up an ppointment as Civil Engineer, under the Colombo lan, to the Government of Ceylon.

INSTITUTION AWARDS

The following awards have been made for papers read efore the Institution and at the Branches during the ession 1951-52 :—

ONDON (HEADQUARTERS) PRIZE

Mr. F. R. Bullen, for a paper on " Unusual Design for Large Constructional Shop."

LANCASHIRE AND CHESHIRE BRANCH PRIZES

Mr. A. Bolton, for a paper on " A New Approach to the Elastic Analysis of Two Dimensional Rigid Frames."

Professor J. A. I. Matheson, for a lecture on " Plasticity and Structural Design,"

MIDLAND COUNTIES BRANCH PRIZE

Mr. F. Brooksbank, for a paper on " Designing for Welding."

NORTHERN COUNTIES BRANCH

Mr. G. S. Gowland, for a paper on " Impressions of the Structural Trade in the U.S.A."

NORTHERN IRELAND BRANCH

Mr. A. H. K. Roberts, for a paper on " Some Practical Applications of Prestressed Concrete."

WESTERN COUNTIES BRANCH

Mr. J. A. Newton, for a paper on " The Civil and Structural Engineers' Contribution towards the Reconstruction of Stapleton Road Gas Works."

CODE OF PRACTICE 112

" THE STRUCTURAL USE OF TIMBER IN BUILDINGS "

The Council for Codes of Practice has now issued in final form, Code 112, " The Structural Use of Timber in Buildings," with which is incorporated Sub-Code 112.100, "Preservative Treatments for Timber Used in Buildings," prepared for the Council by a committee convened by the Institution of Structural Engineers.

The Code deals with the design, fabrication and erection of structural timber work. It is based on the principles of engineering design and on data established by research.

It gives recommendations for the quality of materials, the moisture content of timber for various purposes and the permissible stresses appropriate to particular conditions of loading and service. Design data for the use of nails, screws, bolts, connectors and adhesives are also included.

Recommendations are given for a method of test for components or assemblies of unconventional design which may not be readily amendable to detailed analysis.

Sub-Code 112.100 which is incorporated in the main Code, deals with the preservation of timber by means of wood preservatives against fungal and insect attack, but not with protection against fire or mechanical damage, nor with the methods of minimising the decay hazard of untreated timber by means of seasoning, ventilation and painting.

Copies of the Code (including the Sub-Code) may be obtained from the British Standards Institution, 24/28, Victoria Street, London, S.W.1, price 10s. 6d., post free.

Copies of Sub-Code 112.100 are also issued separately, price 3s., post free.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held on Tuesday, April 14th, at 11, Upper Belgrave Street, London. S.W.1, at 6 p.m., when the following films will be shown by courtesy of the Aluminium Development Association:—

" This is Aluminium," " Fabrication of Aluminium Alloys " and " Heat Treatment of Aluminium."

Hon. Secretary : C. Allen Brown, 43, Coolgardie Avenue, Highams Park, London, E.4.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The Branch Annual Dinner and Ladies' Evening was held at the Longford Hall, Stretford, on February 3rd,

1953, and was attended by approximately 120 members and guests. The Chairman, Mr. W. Bates, M.I.Struct.E. and Mrs. Bates, were supported at the Reception by the President of the Institution, Mr. E. Granter, B.Sc., M.I.C.E., M.I.Struct.E., and Mrs. Granter, and Dr. P. F. R. Venables, B.Sc., F.R.I.C., President of the Royal Institute of Chemists, and Principal of the Royal Technical College, Salford, and Mrs. Venables.

At the conclusion of the Dinner, the Loyal Toast was proposed by the Chairman. Dr. Venables proposed the health of the Institution and Branch, to which responses were made by the President and Chairman. Professor J. A. L. Matheson, M.B.E., M.Sc., M.I.C.E., Vice-Chairman, proposed the health of the Guests, to which Mr. McLellan, of the Manchester Association of Engineers, replied. At the conclusion of the speeches dancing took place until 1 a.m., Mr. C. F. Davies, A.M.I.Struct.E., acting as M.C. Light refreshments were served and it was unanimously agreed that a very successful function had been held.

The following meeting has been arranged :—

Wednesday, April 29th, 1953

Annual Business Meeting. Films : " British Steel " and " A Plan Takes Shape," at the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, April 14th, 1953

Mr. V. H. Lawton (Member), on " Some Notes on Designing to B.S.449," at the Welbeck Hotel, Nottingham, at 7 p.m.

Friday, April 24th, 1953

Annual General Meeting, followed by paper by Mr. O. W. Jones, B.Sc., A.M.I.C.E., (Member), on " Reinforced Concrete Foundations and Structures for Blast Furnaces, and Materials and Handling Plant at Shotton, Nr. Chester," at the James Watt Memorial Institute, Birmingham, at 6 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES BRANCH—GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES BRANCH

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., " Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

The following meeting has been arranged :—

Tuesday, April 14th, 1953

Annual General Meeting, at the Ca'doro Restaurant, Union Street, Glasgow, at 6 p.m.

Hon. Secretary: D. G. Drummond, B.Sc., M.I.Struct.E., A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, M.I.Struct.E. c/o Messrs. T. L. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, April 14th, 1953

Mr. J. R. M. Poole, on " Recent Developments in Timber Structures," at the South Wales Institute of Engineers, Park Place, Cardiff, at 6.30 p.m.

Friday, April 17th, 1953

Mr. R. G. Braithwaite, M.I.C.E., on " Electric Screw Piling," in the County Buildings, Rhiw Road, Colwyn Bay, at 6 p.m.

Wednesday, May 13th, 1953

Annual General Meeting at the Mackworth Hotel, Swansea, at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

The following meeting has been arranged :—

Friday, April 10th, 1953

Annual General Meeting, followed by film show and/or papers presented by Junior Members, in the Geology Lecture Theatre, University of Bristol, at 6 p.m. Tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 39 Effingham Road, St. Andrew's Park, Bristol, 6.

YORKSHIRE BRANCH

The Fourth Ordinary Meeting of the Yorkshire Branch was held on Wednesday, January 21st, 1953, at 6.30 p.m., in the Great Northern Hotel, Leeds, and was attended by about 50 members and visitors. The Chairman, Mr. D. R. S. Wilson, introduced the speaker Mr. Donovan H. Lee, B.Sc., M.I.C.E., M.I.Struct.E. M.I.Mech.E., M.Am.Soc.C.E., who gave a very interesting paper on " Design of Prestressed Concrete," illustrating his talk with a film and lantern slides. Numerous questions and discussion followed, and the meeting closed with a vote of thanks to the speaker.

The following meetings have been arranged :—

Friday, April 17th, 1953

Annual Dinner and Dance at the Parkway Hotel, Bramhope, Leeds, at 7 p.m.

Wednesday, April 22nd, 1953

Annual General Meeting, followed by Mr. J. Dosson M.I.C.E., A.M.I.Mech.E., (Member), on " Sea Defence Works," at The University, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E. A.M.I.Struct.E., P.O Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, City Hall, Johannesburg . 'Phone 34-1111 Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E P.O. Box 1692, Cape Town.

Plastic Design of Plane Frames for Minimum Weight

By Jacques Heyman, M.A., Ph.D.

Summary

A paper[1] presented recently by Professor J. F. Baker s shown the essential simplicity and economy of the astic method of design of rigid framed steel structures. e method has been supported by results from tests th on miniature[2] and on full-scale frames[3], and in ew of the ease with which it can be applied, it is ought to be of interest to publish results of a theoretical vestigation into the material consumption of such mes. In fact, an earlier paper[4] has dealt with the oblem of the exact determination of cross-sections in der that minimum weight design might be achieved ; wever, the solutions were derived by means of sets linear inequalities for the quantities required, in- lving heavy computational labour. The present paper andons the idea of obtaining exact solutions ; instead, is shown that upper and lower bounds may be found r the minimum material consumption, and that these unds may be made sufficiently narrow for practical rposes.

Introduction

The basic ideas of plastic design have already been ven in detail by Professor Baker[1], and these are mmarised below. This author considers also the mplications arising from the effect of axial loads and ear forces on a member subjected mainly to bending ; the present purposes, and to simplify the presenta- n, it will be assumed that these effects are negligible, hough they can be incorporated readily in the theory. addition, it will be supposed that each member is equately supported laterally so that instability of y kind cannot occur ; thus, failure takes place only bending.

The full plastic moment of any section is defined as e maximum bending moment that the section can rry ; it has the property that at any location in a ucture where such a moment exists, rotation can ke place under that (constant) moment. The moment- rvature relationship will thus be as shown in Fig. 1,

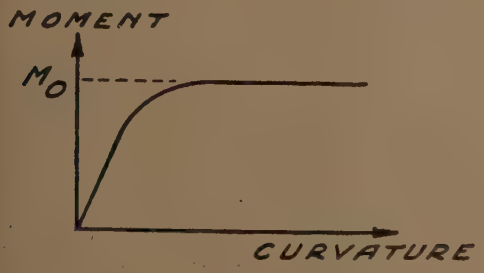


Fig. 1

ere the full plastic moment has the value M_0 : the int in a structure at which M_0 is attained is known a plastic hinge. Collapse of a redundant structure kes place when a sufficient number of plastic hinges ist to transform the whole or a part of the structure to a mechanism of one degree of freedom.

The problem discussed in this paper may be stated as llows : given the geometrical configuration of a frame ich is acted upon by specified fixed loads, how should e various members be proportioned so that the total weight of the structure is as small as possible ? It will be assumed that some load factor (a constant multiplier) is incorporated in the values of the loads so that although the frame is designed to collapse, normal working conditions will obtain when the loads have a fraction only of their design values.

In order to develop suitable methods for design, it will be assumed that a continuous range of sections is available, so that a section can be chosen with any specified full plastic moment. The total material consumption (X) of a structure will be taken as

$$X = \Sigma M_n l_n \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where l_n is the length of the nth member of a structure whose full plastic moment is M_n.

This linearised relationship will be used only as a tool for the determination of the various values M_n in the different portions of the structure. It has been shown[4] that the beam and stanchion sections determined in this way for single-bay portal frames under several conditions of loading correspond exactly to the results obtained by a more exact analysis[5] using a practical relationship. In fact, any plastic moment/ weight curve of roughly the right " shape " will lead to minimum weight sections that are exactly or very nearly correct.

Upper and Lower Bounds on Collapse Loads

Before proceeding to the minimum weight design of frames, it will be useful to state the theorems which have been proved[6] for the analysis of given frames. Since this paper is concerned with non- varying loads, it is convenient to specify the system of loads as ratios of one of their number, so that the term " collapse load " implies the whole system.

An Upper Bound for the Collapse Load of a Given Frame

If a number of hinges is inserted arbitrarily into the frame in order to turn it into a mechanism of one degree of freedom, then a virtual work equation may be written in which the work done by the load during a small displacement of the mechanism is equated to the work dissipated in the hinges during the same displacement. The value of the load resulting from this equation is always either greater than the true collapse value, or, at most, equal to the true value if the actual mode of collapse has been chosen by accident. Thus an upper bound may be derived.

A Lower Bound for the Collapse Load of a Given Frame

Suppose a value of the load can be discovered for which it is possible to construct an equilibrium bending moment distribution that nowhere in the frame violates the collapse condition ; that is, the bending moments at each section must satisfy not only the independent equations of equilibrium, but must also be less than, or at most equal to, the fully plastic moments at those sections. Then the value of the load corresponding to this equilibrium system is always less than, or, at most, equal to, the true collapse value. Thus a lower

bound may be derived. For the application of this theorem, it is not necessary that the bending moment distribution be the actual one that would occur under the load ; any equilibrium distribution which does not violate the collapse condition will give a lower bound.

[It may be noted that in the application of the first theorem the assumed bending moment distribution corresponding to the plastic hinges will not satisfy in general the equilibrium conditions, so that attention is concentrated only on the kinematic behaviour of the frame ; while in the application of the second theorem, the compatibility conditions will not usually be satisfied, attention being concentrated only on the equilibrium equations. Both conditions of course are satisfied for the true value of the collapse load.]

Upper and Lower Bounds on Minimum Weight

The theorems for bounds on minimum weight follow from the two theorems stated above.

A Lower Bound for the Minimum Weight

Consider a frame to be divided into two parts by some arbitrary dividing line. The two parts of the frame will be referred to as parts A and B. Now suppose that part B of the frame is " frozen," that is, the members within that part are made so strong that there is no possibility of their being concerned in the collapse of the frame as a whole. Suppose now that part A is designed to collapse under the given design load, and that in addition, the members comprising part A are so proportioned that minimum material consumption is achieved for this part alone. Let W_A be the weight of part A, and W_B the weight of part B determined by considering the collapse of part B alone with part A " frozen." Then the minimum weight W of the frame as a whole is greater than the sum of the minimum weights of the two parts determined in this way. An immediate corollary of this theorem, which may be proved by successive applications, is that one or both of the two parts may be subdivided into component parts, so that in fact the original frame may be subdivided into as many component parts as is considered necessary.

Proof : Suppose the sections derived by dividing the frame into two parts are expressed as multiples of the section of one member of the frame whose full plastic moment is M_o. Now portion A of the frame has been designed to collapse under the given load. But the mechanism associated with this collapse mode is a mechanism of collapse of the frame as a whole, and similarly the mechanism for the collapse of part B is a mechanism of collapse of the frame as a whole. Hence the collapse load of the whole frame whose sections are specified in terms of M_o must be less than the given design load, by the upper bound theorem on the load factor given above. In order then to achieve collapse of the whole frame under the design load the section moments M_o must be increased, and the weight of the whole frame becomes greater than $(W_A + W_B)$. It remains to be shown that, although this new weight is not of course the minimum weight of the whole frame, there is no possibility of transferring material from part A to part B in such a way that $W < (W_A + W_B)$. Since W_A is by definition the minimum weight of part A considered alone, any reduction below W_A in the material used for this part of the frame would result in immediate collapse of this part, and similarly for part B. This completes the proof of the theorem.

An Upper Bound for the Minimum Weight

Suppose any arbitrary bending moment distribution is determined that is in equilibrium with the given design load, and that section moments are assigned to the various members such that the collapse condition for any member is not violated. Then the weight of the frame corresponding to these sections is an upper bound on the minimum weight of the frame.

Proof : Suppose the sections derived from the equilibrium bending moment distribution are expressed as multiples of the section of one member of the frame whose full plastic moment is M_o. By the lower bound theorem on the collapse load for a given frame, the actual collapse load of the frame must be greater than the design load. Hence the sections of the frame must be reduced in order that collapse should occur under the design load, so that the weight of the frame determined in the above way must be greater than the minimum weight. This completes the proof of the theorem.

Applications

Example 1. It is required to design the simply supported beam, with an intermediate support, carrying the loads shown in Fig. 2. The full plastic moment of

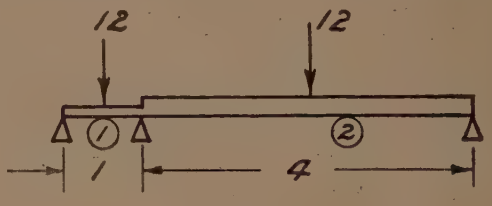


Fig. 2

the beam used in the left-hand span will be denoted by M_1, and that of the beam used in the right-hand span by M_2. Equation (1) for the material consumption may be written

$$X = M_1 + 4\,M_2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (2$$

The minimum material consumption of the system as a whole will be denoted by W.

Suppose the left-hand beam to be made so strong that collapse occurs only in beam 2. This latter beam can be designed as the propped cantilever shown in Fig. 3. It is clear for this simple system that the collapse mechanism is as shown in Fig. 4, where the rotation at

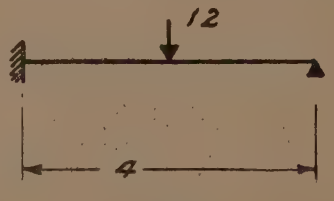


Fig. 3

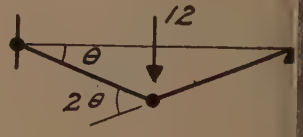


Fig. 4

the left-hand hinge is θ and at the centre hinge 2θ. The total work dissipated in the hinges is thus $3\theta M_2$. The deflection of the load is 2θ, so that the work done during the displacement is 24θ ; equating these two values gives immediately that $M_2 = 8$. Similarly, the minimum value of M_1 is $M_1 = 2$. Hence from Equation (2)

$$W \geq X = 34 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3$$

For the determination of an upper bound on W, consider the collapse of the system shown in Fig. 5. The moment of the left-hand span has been taken as M, and that on the right as $4M$, corresponding to the ratios found for the lower bound. It is easily shown that the collapse mechanism is that given in Fig. 6, and by the equation of virtual work

$$12.2\theta = M.\theta + 4M.2\theta$$

$$\text{i.e., } M = \frac{8}{3} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4$$

The collapse bending moment distribution corresponding to this value of M is shown in Fig. 7. Now this distribution is in equilibrium with the applied loads, and, furthermore, although plastic hinges are formed at two sections of the beam, the collapse condition is nowhere *exceeded*. The distribution, in fact, satisfies the conditions for an

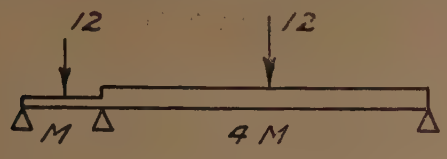


Fig. 5

upper bound, and the total weight of the beams corresponding to $M_1 = 8/3$, $M_2 = 32/3$ is greater than the minimum weight of the system. Hence

$$W \leq X = 45{\cdot}3 \quad \ldots\ldots\ldots\ldots \quad (5)$$

or, combining Equations (3) and (5)

$$45{\cdot}3 \geq W \geq 34{\cdot}0 \quad \ldots\ldots\ldots\ldots \quad (6)$$

This technique of using a kinematic solution in order to obtain an equilibrium solution will be used in the later examples. It will be noted that an arbitrary ratio of 4

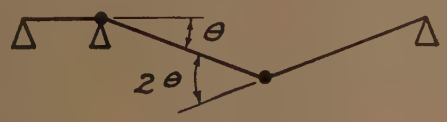


Fig. 6

was taken for the two full plastic moments of the spans ; any other ratio will give a different value for the upper bound. Suppose that the full plastic moment of the right-hand span is aM_0, that of the left being M_0. Then

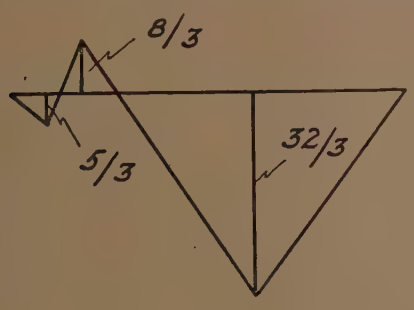


Fig. 7

if collapse occurs by the same mechanism (Fig. 6), it is easily shown that

$$M_0 = \frac{24}{1 + 2a} \quad \ldots\ldots \quad (7)$$

In addition, the material consumption, from Equation (2), is given by

$$X = M_0(1 + 4a) \quad \ldots\ldots\ldots\ldots \quad (8)$$

so that, eliminating M_0,

$$X = 24\left\{\frac{1 + 4a}{1 + 2a}\right\} \quad \ldots\ldots \quad (9)$$

From this equation, for X to be as small as possible, the ratio a should be reduced to its lowest possible value. But the smallest value that a can take for failure to occur by the assumed mechanism is $a = 1$, since it was assumed in deriving Equation (7) that the hinge at the intermediate support was formed in the left-hand span. With this value of a, the value of X given by Equation (9) becomes $X = 40$, so that

$$W \leq 40 \quad \ldots\ldots\ldots\ldots \quad (10)$$

To complete the investigation of the variation of the ratios of the two spans, it can be shown that for $a \leq 1$, with the hinge at the support occurring in the right-hand span, the material consumption is given by

$$X = \frac{8(1 + 4a)}{a} \quad \ldots\ldots\ldots\ldots \quad (11)$$

For the minimisation of X, a should in this case be made as large as possible, that is, $a = 1$, and the value of X is again 40. A uniform beam in the two spans is in fact the best design for this particular example, and the solution of the problem is $W = 40$.

Discussion of the Application of the Method

It will be seen in the above example that the lower bound on the minimum weight, although appreciably below the final value for W, cannot be improved by successive applications of the theorem. There is only one way in which the system can be divided into two parts, and that is by a dividing line at the intermediate support. The upper bound, however, can be reduced by stages, since there are an infinite number of possible equilibrium bending moment distributions. The situation is exactly analogous to the determination of the collapse load for a given frame ; the bound given by the construction of a mechanism cannot be improved indefinitely, as there are only a finite number of possible mechanisms, whereas the bound derived from an equilibrium solution can be made to coincide with the true value of the quantity required. For this problem of the continuous improvement of the bound obtained from a mechanism, the author and Nachbar[7] have suggested the use of mechanisms of many degrees of freedom, and have shown that the upper bound on the collapse load can in fact be made to coincide with the true collapse load. More recently, Neal and Symonds[8] have considered the combination of a large number of independent local collapse mechanisms ; this method gives a very quick and systematic method of improving the kinematic solution. (It is used below as a step in the determination of the true mode of collapse for another example in minimum weight design.) The problem of improving the bound becomes less important in general as the number of redundancies of the frame to be designed increases. Since there are a large number of mechanisms of failure of a highly redundant structure, it is likely that one at least of these will give a bound that is very close to the true solution. It is probable that a method for the improvement of the lower bound in minimum weight design would take place along one or both of the lines mentioned above.

The general technique for the minimum weight design of beams and plane frames recommended in the present paper may be tabulated as follows :

1. Divide the frame into as many separate parts as possible, and calculate the minimum weight for each part. The sum of these weights will give a lower bound on the minimum weight of the whole frame.

2. This bound will be in all probability rather crude. For its improvement, it should be noted that if the frame is divided into fewer and larger parts, the sum of the minimum weights must be increased. Hence the possibility of combining several small parts of the frame should be explored, if this can be done in such a way that the calculation of the minimum weights of the various parts is not made too laborious.

3. The section moments determined in the solution for the lower bound will probably be in roughly the right ratios for the final solution. Hence, using these ratios of the full plastic moments for the various components

of the frame, a frame may be designed which will just collapse under the given design loads. The resulting weight of the frame is then an upper bound on the minimum weight.

4. The upper bound may now be lowered. Specifying the full plastic moments of the members of the frame as ratios of the full plastic moment of one member, and investigating the variation of one member at a time, an equation similar to Equation (9) may be written. This equation will immediately indicate whether the size of the member under consideration should be increased or decreased. The writing of the weight equation implies that the mode of collapse is unchanged by the variation in size of the member ; this fact will give limits to the range of variation possible. On the introduction of a new mode of collapse, the weight equation must be rewritten and the process repeated. By the consideration of each member of the frame in turn, the upper bound may be continually decreased.

Example 2. It is required to design the single-storey, two-bay frame shown in Fig. 8. The two beams

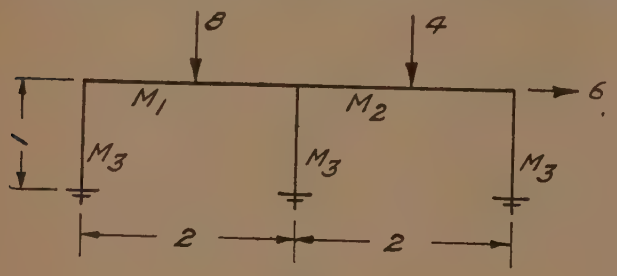


Fig. 8

have different sections M_1 and M_2 and the three stanchions have the same full plastic moment M_3.

1. The frame may be divided into three parts shown as the independent mechanisms in Fig. 9. As these

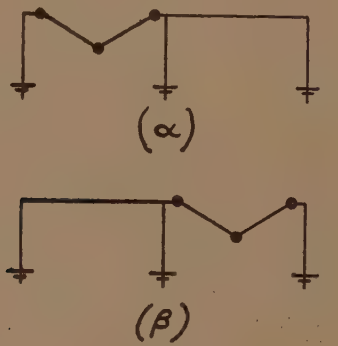


Fig. 9

mechanisms correspond to those required in the method described by Neal and Symonds[8], it will be convenient to tabulate the appropriate calculations. The values of the moments required for each mechanism are $M_1 = 2$, $M_2 = 1$, $M_3 = 1$, determined as follows from the virtual work equation :

$$\left.\begin{array}{ll}(\alpha) & 4M_1 = 8 \\ (\beta) & 4\,M_2 = 4 \\ (\gamma) & 6\,M_3 = 6\end{array}\right\} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

In each case in Equations (12), the value of the full plastic moment is found by equating the work dissipated in the hinges to the work done by the applied loads, the rotation θ being a common factor of both sides.

Now the material consumption of the whole frame is given by

$$X = 2(M_1 + M_2) + 3(M_3) \quad . \quad . \quad . \quad . \quad (13)$$

For the minimum values of the members given above,

$$X = 9$$

$$\text{so that } W \geq 9 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (14)$$

2. A way of increasing the size of the component parts of the frame is to combine the left-hand beam with the stanchions, leaving the right-hand beam to be designed separately as $M_2 = 1$. Suppose that failure of the larger portion of the frame occurs by the mode shown in Fig. 10 (beam M_2 is taken to be rigid, so that a hinge cannot be formed in its length). If $M_1 = aM$, $M_3 = M$, then the equation of virtual work becomes

$$14 = M(5 + 3a) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (15)$$

The material consumption for the stanchions and the left-hand beam is given from Equation (13) as

$$x = 2M_1 + 3M_3$$

$$\text{or } x = M(3 + 2a) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (16)$$

so that, using Equation (15)

$$x = \frac{14(3 + 2a)}{(5 + 3a)} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (17)$$

For x to be as small as possible, a should be decreased. The minimum value of a is governed by the condition that the collapse criterion should not be exceeded at the left-hand end of the beam. The minimum value of a is determined from this condition as 13/9, with $M = 1.5$, so that the minimum value of x is

$$x = 8.8 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (18)$$

$$\text{i.e., } W \geq X = 10.8 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (19)$$

3. For the determination of an upper bound, assume in the first instance that the members have full plastic moments given by $M_1 = 2M$, $M_2 = M$, $M_3 = M$. Combining the three mechanisms in Fig. 9 in such a way that the collapse load is a minimum, the collapse mode shown in Fig. 10 is found to be correct. The corresponding value of M can be determined by combining (α) and (γ) in Equations (12) :

$$4(2M) + 6(M) - (M + 2M) = 8 + 6 \quad . \quad (20)$$

The subtraction of $3M$ from the left-hand side in Equation (20) is due to the rotation of the joint at the left-hand end of beam M_1. Thus, M is found to have the value 14/11, and an upper bound may be written for the weight ; using Equation (13), the bounds are

$$11.5 \geq W \geq 10.8 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

4. In order to improve the upper bound, suppose that $M_1 = aM$, $M_2 = bM$, and $M_3 = M$. If the collapse mechanism is the same, Fig. 10, the equation of virtual

Fig. 10

Fig. 11

work is again given by Equation (15), and the material consumption is given by

$$X = M(3 + 2a + 2b) \quad . \quad . \quad . \quad . \quad . \quad (22)$$

$$\text{or } X = 14\left\{\frac{3 + 2a + 2b}{5 + 3a}\right\} \quad . \quad . \quad . \quad . \quad . \quad (23)$$

quation (23) shows immediately that b should be duced in value : but any reduction in the value of b mediately changes the mode of collapse of the frame that shown in Fig. 11. The equation of virtual work ust be written again for this new mode,

$$14 = M(5 + 2a + 2b) \quad . . . . . (24)$$

id combining with Equation (22)

$$X = 14\left\{\frac{3 + 2a + 2b}{5 + 2a + 2b}\right\} \quad . . . . . (25)$$

his last equation shows that b should still be reduced, as ould a. However, the variation of a and b will be vestigated separately. Keeping a fixed at the value 2, may be decreased until the full plastic moment is just ached at the centre of the right-hand beam. The lution of the various equations gives

$$\left.\begin{aligned} b &= 3/4 \\ M &= 4/3 \end{aligned}\right\} \quad . . . . . (26)$$

is noticed that $bM = 1$, that is, the minimum possible ction is being used for the right-hand beam. Keeping ie full plastic moment of this beam fixed at the value 1, ie value of a may be reduced until a third hinge is rmed at the joint between the two beams. At this ndition,

$$\left.\begin{aligned} a &= 17/10 \\ b &= 7/10 \\ M &= 10/7 \end{aligned}\right\} \quad . . . . . (27)$$

id the location of hinges in the frame is as shown in g. 12. If a is lowered further, the mode again changes that shown in Fig. 13. For this mechanism, a should

Fig. 12

Fig. 13

increased in order to achieve minimum weight ; the lues given in Equations (27) are in fact the best design r the frame. Substitution into Equation (22) gives

$$X = W = 11.1 \quad . . . . . (28)$$

Example 3. As a final example of the method, the frame in Fig. 14 will be designed for

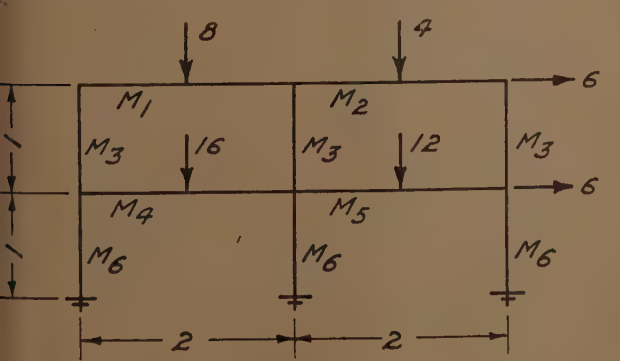


Fig. 14

inimum weight. The four beams all have different ctions, and the stanchions at each floor level are entical. Crude bounds only will be derived, although ese may be narrowed with very little labour.

If the collapse of each beam and the stanchions is considered separately, the following minimum section moments are required :

$$\left.\begin{aligned} M_1 &= 2 \\ M_2 &= 1 \\ M_3 &= 1 \\ M_4 &= 4 \\ M_5 &= 3 \\ M_6 &= 2 \end{aligned}\right\} \quad . . . . . (29)$$

The total weight of the frame can be expressed as

$$X = 2M_1 + 2M_2 + 3M_3 + 2M_4 + 2M_5 + 3M_6 \quad (30)$$

so that

$$W \geq X = 29 \quad . . . . . (31)$$

This bound can be improved by increasing the size of the component parts of the frame. If M_4, M_5, and M_6 are calculated separately to have the values given in Equations (29), but the remaining members combined together as one unit, then the immediate problem becomes one of the design of the single-storey, two-bay portal formed by the top half of the frame. Now this subsidiary frame has already been designed in Example 2, and was shown to have a minimum weight of 11.1. Thus a better design to give a lower bound for the whole frame is

$$\left.\begin{aligned} M_1 &= 17/7 \\ M_2 &= 1 \\ M_3 &= 10/7 \\ M_4 &= 4 \\ M_5 &= 3 \\ M_6 &= 2 \end{aligned}\right\} \quad . . . . . (32)$$

and the new lower bound is

$$W \geq 31.1 \quad . . . . . (33)$$

An analysis of the frame as a whole shows that all these values of the moments should be increased by a factor 1.23 to give a design which is on the point of collapse, so that W lies between the limits

$$38.4 \geq W \geq 31.1 \quad . . . . . (34)$$

The effect of dividing the frame into a small number of large parts, that is, parts containing several members, is to turn the analysis of a complicated frame into the analysis of a number of smaller and more simple frames.

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Prestressed Concrete Bridges and Other Structures

Discussion on Mr. Donovan H. Lee's Paper*

The CHAIRMAN introduced Mr. Lee, who then gave a brief summary of the paper and showed illustrations of the tests of beams at Port Tampa for the Tampa Bay Bridge in Florida (Fig. 31), a test of a beam for the Leyton-Marshes Culvert, which was made by the Cement and Concrete Association (Fig. 32), the first prestressed concrete overline railway bridge in Eire (Fig. 33), and some views of the 160 ft. span prestressed concrete railway bridge at Rotherham.

The CHAIRMAN, whose pleasure it was to propose a vote of thanks to Mr. Lee for his very able paper and his interesting slides and explanations, said the interest in prestressed concrete construction was indicated by the fact that the lecture-hall was filled, although it was only a fortnight before Christmas. The large attendance was also a compliment to the lecturer.

One was glad, the Chairman continued, that Mr. Lee had made some reference to costs, for all engineers were interested in costs as well as in technicalities, and he hoped that speakers in the discussion would give some idea, when describing works in prestressed concrete, of how the costs compared with normal construction.

In presenting the paper, Mr. Lee had been very fair ; he had explained that, although he was presenting a paper on prestressed concrete, it did not necessarily mean that in his view that form of construction should be used everywhere.

Congratulating him on his association with the Tampa Bay bridge, the Chairman said it was something of which he might very well be proud.

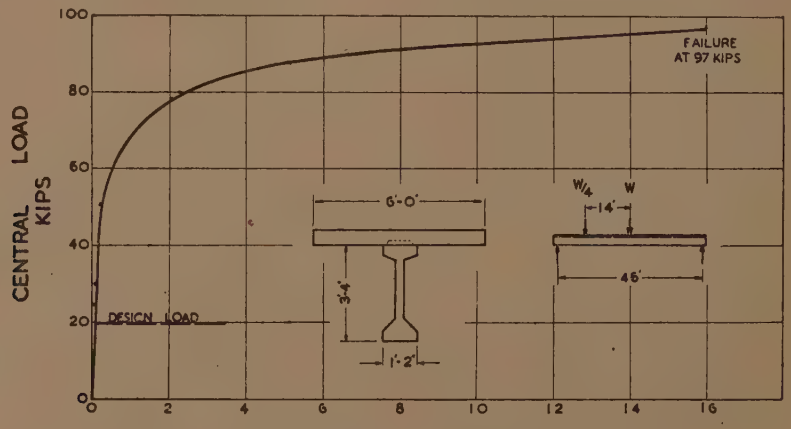
With regard to Fig. 21, showing a typical cross-section of a steel girder bridge where prestressed concrete decking was used instead of a steel deck, the President asked whether there was a saving in cost as a result of the adoption of that form of construction, or whether the saving effected was in steel only. He was particularly interested in the latter part of the paper, for some of the work described was of very considerable importance, especially as there is a shortage of steel and engineers are exhorted by Government Departments to save steel.

Lieut.-Colonel R. F. GALBRAITH, M.C. (Vice-President) first thanked Mr. Donovan Lee for his most interesting paper.

Suggesting that Mr. Lee should emphasise the fact that the system using " Macalloy " bars and Lee McCall high efficiency nuts was British both in conception and manufacture, he felt we should be proud of that fact when we considered the examples that Mr. Lee had shown.

Some economic factors were discussed in the paper, but Col. Galbraith pointed to two additional ones which assisted towards economy. The first was the need to standardise units and sections in order to obtain the maximum re-use of forms or moulds. The Leyton Marsh culvert and the Lower Tampa Bay bridge were excellent examples of repetition. The second factor was the need for the education of contractors in the techniques of prestressing. At the present time there was a tendency to quote from £2 to £6 each for tensioning bars, and the price bore no relation to the cost of performing the operation. The contractor obtained what he could for the job ; but if sufficient competition came into play, the price would come down nearer to the actual cost of the operation, and we should achieve progress in prestressed work. Recently he had watched three men with two jacks tension 30 pairs of bars and secure the nuts in about seven hours. We could work out the cost from that.

Mr. Lee had discussed the use of " slip-rods " and favoured wrapping rather than coating. As an alternative, Col. Galbraith had found recently a tube made of aluminium foil and bitumenised paper, which could be slipped over the bar, thus eliminating wrapping. A



Test on Composite Beam for Tampa Bay Bridge

Deflection—Inches

Fig. 31

Presented to the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, December 11th, 1952, at 6 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, *Vol.* XXX, *No.* 12, *p.* 302.

cost of about 6½d. per ft. for a 1⅛ in. dia. bar, that was obably the cheapest method of dealing with "slip-ds."

Col. Galbraith illustrated some additional uses of the Macalloy" system at the Plashet Grove County condary School for Girls, at East Ham.

Slide 1 showed a group of bars ready for wrapping ior to fixing in a retaining wall. The bars were bent to the form of a "U," the straight arms were greased id wrapped with one-ply roofing felt and then cast *situ* in the concrete wall. The bars were of high nsile steel.

Slide 2 showed a series of vertical and horizontal bars the bottom of a retaining wall, fixed in position ready

Slide 7. Another picture of the window unit (with the forms stripped); ready for tensioning.

Mr. LEE, replying to the President, said Fig. 21 showed a cross-section through precast prestressed slabs which formed the decks of plate girder underline rail bridges. For the photograph in Fig. 22 he was indebted to Mr. P. S. A. Berridge, who was responsible for the bridges of the Western Region of British Railways; in the course of the evening he thought he would need to become indebted to Mr. Berridge for answering questions on the design of them as he did not know the answer to the President's question as to costs.

Commenting on Col. Galbraith's remarks, he said he had never had the pleasure of seeing the contract that

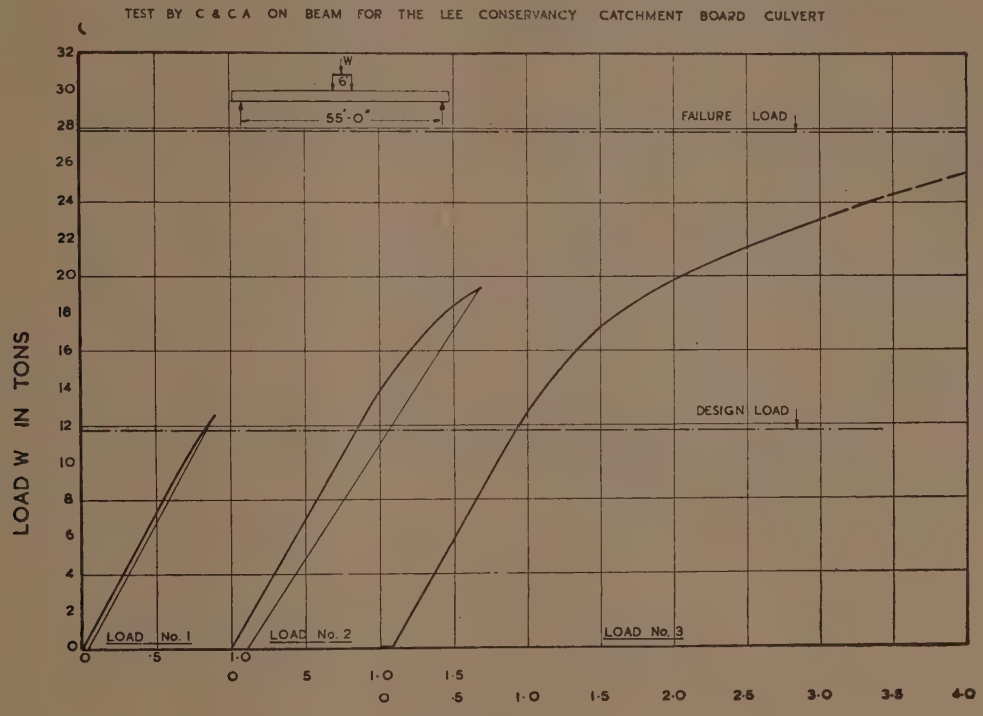


Fig. 32

r concreting. The bars were wrapped, and a strip of eavy expanded metal was inserted in the "U" to istribute the high local stresses.

Slide 3 showed a length of the retaining wall with the ertical bars in position and projecting from the concrete e. The 9 in. brick wall acted as external formwork. he inside surface of the brick wall was coated with arine cement as an additional precaution against amp. The tops of the rods were threaded, and the uts had been removed.

Slide 4. A length of wall completed and partially ack filled with the "Macalloy" bars shown projecting ady for tensioning.

Slide 5. Part of a concrete window-frame 20 ft. long y 10 ft. high, forming a wall of a classroom. The four embers of the unit were precast and post-tensioned to ake a "monolithic unit."

Slide 6. Part of the mould for the window-frame ith two "Macalloy" bars in the base of the frame. here was secondary reinforcement at the corners. he mullions were cast separately and inserted into the rm before the remainder was cast.

he had mentioned; but he felt that Col. Galbraith had hardly had the opportunity to do it justice in the few remarks he had made and the illustrations he had shown. From what Mr. Lee knew of it, it was truly an example of steel economy, and assuming that the weight of steel in a prestressed building for a school and the retaining walls was in the ratio of 1/2½ as compared with reinforced concrete, it was easy to imagine how much steel might be saved if necessary by the wider use of prestressing. He hoped Col. Galbraith would be able to give the Institution more detailed information on this contract in the near future.

Referring to non-bonded bars, i.e. coated and wrapped bars, he said that in America there had been a leaning towards this, but fully bonded was being used on the Tampa Bay bridge and elsewhere where these high-tensile bars were being used except in the case of hydraulic structures like tanks. It would be realised that non-bonded construction did convey to the ends all alterations and increments of tension in the prestressing elements even though they are usually very small. This was not a good thing. It should also not be overlooked that the ultimate load is lower with non-bonded than with fully bonded construction. However, instead

of coating a bar it can be wrapped with corrugated paper and bitumen paper and he had used this method recently in this country where bond was not important. For bonded construction flexible tubing made of bitumenised paper and aluminium foil or paper reinforced by a steel

Fig. 33

wire spiral were very convenient as the bar was placed before concreting. Both give good bond when the annular space around the bar is grouted after stressing.

Mr. P. S. A. BERRIDGE (British Railways, Western Region), as a visitor, thanked the Institution for having permitted him to join in the discussion and said that to be given the opportunity to speak on Mr. Lee's excellent paper was an honour he much appreciated.

Referring to Fig. 18, illustrating the strengthening of a triangulated girder carrying a road over the railway near Snow Hill Station, Birmingham, he said the location was a very smoky one, and the flanges of the bottom chord—which, incidentally, had been filled with concrete, making strengthening by the addition of mild steel difficult—had suffered a good deal from corrosion. It was the only really weak member in the girder. That girder was very nearly parallel with the tracks below and the road was a busy one, so that the complete renewal of the girder would have been extremely difficult. The prestressing would enable the stresses in the bottom chord to be reduced to within permissible limits. Some strengthening of the top chord was being done by introducing three portal frames over the footpath, thus reducing the ratio of effective length to radius of gyration of that compression member.

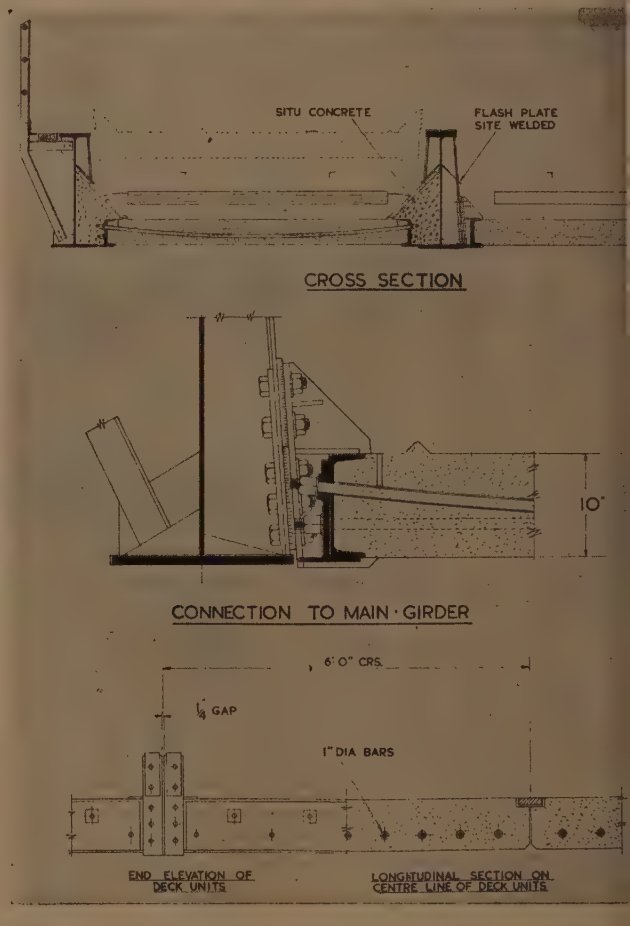
The 150 ft. main girders of a deck type span carrying a single line of railway over the river at Monmouth were to be strengthened in a similar way, but in that case the bars would be painted and not encased in concrete. It was seldom possible, he said, to relieve girders of dead load when adding new mild steel to mak up for the deficiency of section.

He said he was not very clear about Mr. Lee's stat ment on page 309 that " the introduction of any pr stressing steel as part of the load-carrying structu does lead to an increase in the strain, and therefore th deflection, as compared with an equal amount strengthening provided at a lower working stress, say b mild steel." If a girder were not relieved of its dea load, the addition of mild steel in a tension member i which the stresses under live load on the original sectio were already up to the permissible limit would n prevent those stresses being exceeded when the live loa was increased. On the other hand, by prestressing th member, the dead load stresses on the original sectio could be reduced, thus giving a greater reserve strength for the live load.

In the Monmouth bridge, the stresses in tons p sq. in. in the wrought iron of the bottom chords befo and after prestressing were :—

	Dead load	Live load	Total
Before prestressing	3.15	7.00	10.15
After prestressing ...	0.40	6.83	7.23

The permissible stress in that wrought iron wa 7.4 tons per sq. in. There were four 1 in. diamet " Macalloy " bars per girder. In the prestressing, tho were prestressed to 27.2 tons per sq. in. ; and und

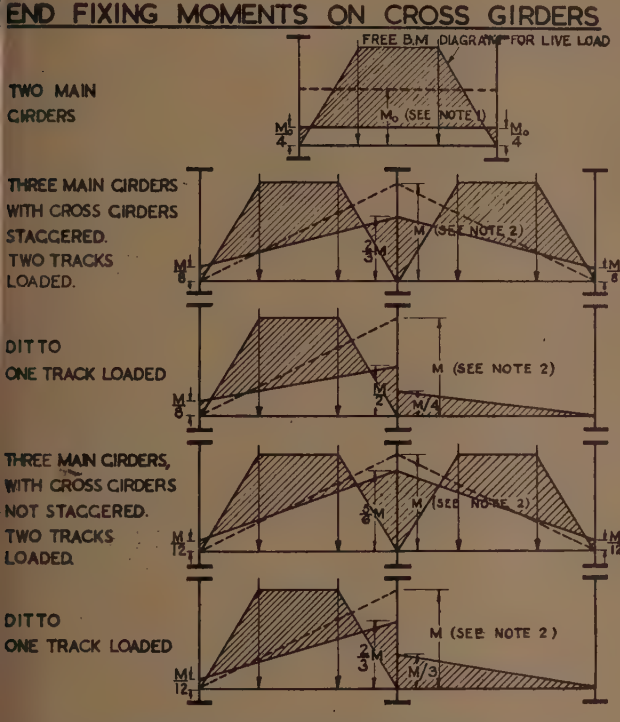


Slide 1

live load there would be an increment of stress of 3.8 tons per sq. in., giving a total stress of just over 31 to per sq. in. The prestressing would add 7/16 in. to th existing 1¾ in. camber, and the deflection under liv load would be 2 in.

For comparatively wide units, as shown in Figs. 21 ıd 22, some transverse mild steel reinforcement was eeded to take the tensile stress induced across the ırface by the longitudinal prestressing. Mr. Lee had rawn attention to those requirements ; and previous xperience on British Railways served to underline that bservation. In one of their earlier designs that rein- ırcement had been left out and a longitudinal crack ad developed under the prestressing load.

On the Western Region there were four bridges of the ype shown in Fig. 21 under manufacture. All were


END FIXING MOMENTS ON CROSS GIRDERS
FREE B.M. DIAGRAM FOR LIVE LOAD
TWO MAIN GIRDERS
Mo (SEE NOTE 1)
THREE MAIN GIRDERS WITH CROSS GIRDERS STAGGERED. TWO TRACKS LOADED.
M (SEE NOTE 2)
DITTO ONE TRACK LOADED
M (SEE NOTE 2)
THREE MAIN GIRDERS, WITH CROSS GIRDERS NOT STAGGERED. TWO TRACKS LOADED.
M (SEE NOTE 2)
DITTO ONE TRACK LOADED
M (SEE NOTE 2)
NOTES:— 1. FOR BRIDGES WITH TWO MAIN GIRDERS, AND WHEN CROSS GIRDER IS SYMMETRICALLY LOADED, Mo = AVERAGE HEIGHT OF FREE B.M. DIAGRAM.
2. FOR BRIDGES WITH THREE MAIN GIRDERS, M = END MOMENT FOR FULL FIXITY AT CENTRE GIRDER. (WHEN LOADING IS SYMMETRICALLY PLACED ON EITHER OR BOTH CROSS GIRDERS, M = 1·5 TIMES THE AVERAGE HEIGHT OF THE FREE B.M. DIAGRAM.)
3. THE B.M. ON THE CROSS GIRDER IS GREATER WITH ONE TRACK LOADED, AND ON THE END CONNECTIONS WITH TWO TRACKS LOADED.
4. WHEN CROSS GIRDERS ARE UNSYMMETRICALLY LOADED, THE END FIXING MOMENTS MUST BE CALCULATED.
5. THE FIXING MOMENTS SHOWN ABOVE MAY BE USED IN DESIGN, TO DETERMINE MAXIMUM STRESSES IN CROSS GIRDERS AND ON THEIR END CONNECTIONS.

Slide 2

quare spans, and the problem of providing the triangular leck units at the ends of a skew span in precast pre- tressed concrete had not been solved ; though he had very confidence that they would eventually solve the roblem. Mr. Berridge showed some details of the nits and their connections to the main girders.

Slide 1. Drawing of cross-section of bridge showing letails of the deck units and the connections to the nain girders. In the connections to the main girders, hear plates welded to the sloping face of the angles on the leck units rested directly on similar shear plates welded to he stiffeners. These shear plates were designed to ake the vertical reaction from the ends of the deck nits while the bolts were designed to carry the stresses nduced by the moment at the joint ; the bolts were not intended to take any shear, although they could well do so.

The advantages claimed for that design were :—

(1) Erection was very simple, the main girders could be landed in their final positions and the deck units dropped in between them without either having to move the girders laterally or to tilt the units. The shear plates on the stiffeners and the ends of the units acted as erection ledges and the bolting was relatively quick.

(2) There was no site riveting or site welding during erection. The small amount of welding necessary to fasten the flash plates to the stiffeners could be carried out after traffic had been restored.

(3) The deck was of crackless concrete and there was no maintenance painting involved underneath, except for the flanges of the main girders.

(4) With a 10 in. deep unit and 3 in. of ballast under the sleepers, the construction depth required was only 2 ft. 1½ in., there being no overall waterproofing.

(5) The units were extremely simple to make and pre- stress, and there was no complicated shuttering.

An important point to bear in mind was that the units and the steelwork must be made at the same works. The sloping contact faces of the steel at the ends of the units and the stiffeners must be a reasonably good fit. A steel jig was used to ensure that the slope of the steel at the ends of the units, as well as the sizes of the units, should be the same for every unit.

In the past, proper attention had not always been said to the design of the connections between the cross- girders and the main girders of small through-type spans. The cross-girders had been generally designed as simple beams, but in making the connections to the main girders a considerable degree of end-fixity had been introduced.

Slide 2 indicated the magnitude of the end-fixing moments on cross-girders, and Mr. Berridge drew attention to the moments at the centre girder when both tracks were loaded. Although the slide was prepared on data obtained from steel cross-girders, it served to indicate the moments which had been allowed for in designing the prestressed concrete deck units.

Slide 3 showed the magnitude of the stress in those units. Full lines indicated the stresses under conditions of full end-fixity, and the dotted lines showed the stresses under 50 per cent. end-fixity at the centre girder. The stress induced on bolting, as shown in the second diagram from the top, was due to the tops of the ends of the units first coming into contact with the stiffeners after the units had hogged while being prestressed. The high strength bolts were tightened to a pre-determined torque and took care of the stresses in that connection. Under full load there was no separation of the contact surfaces, and no loading of the bolts beyond the stresses induced in them by tightening.

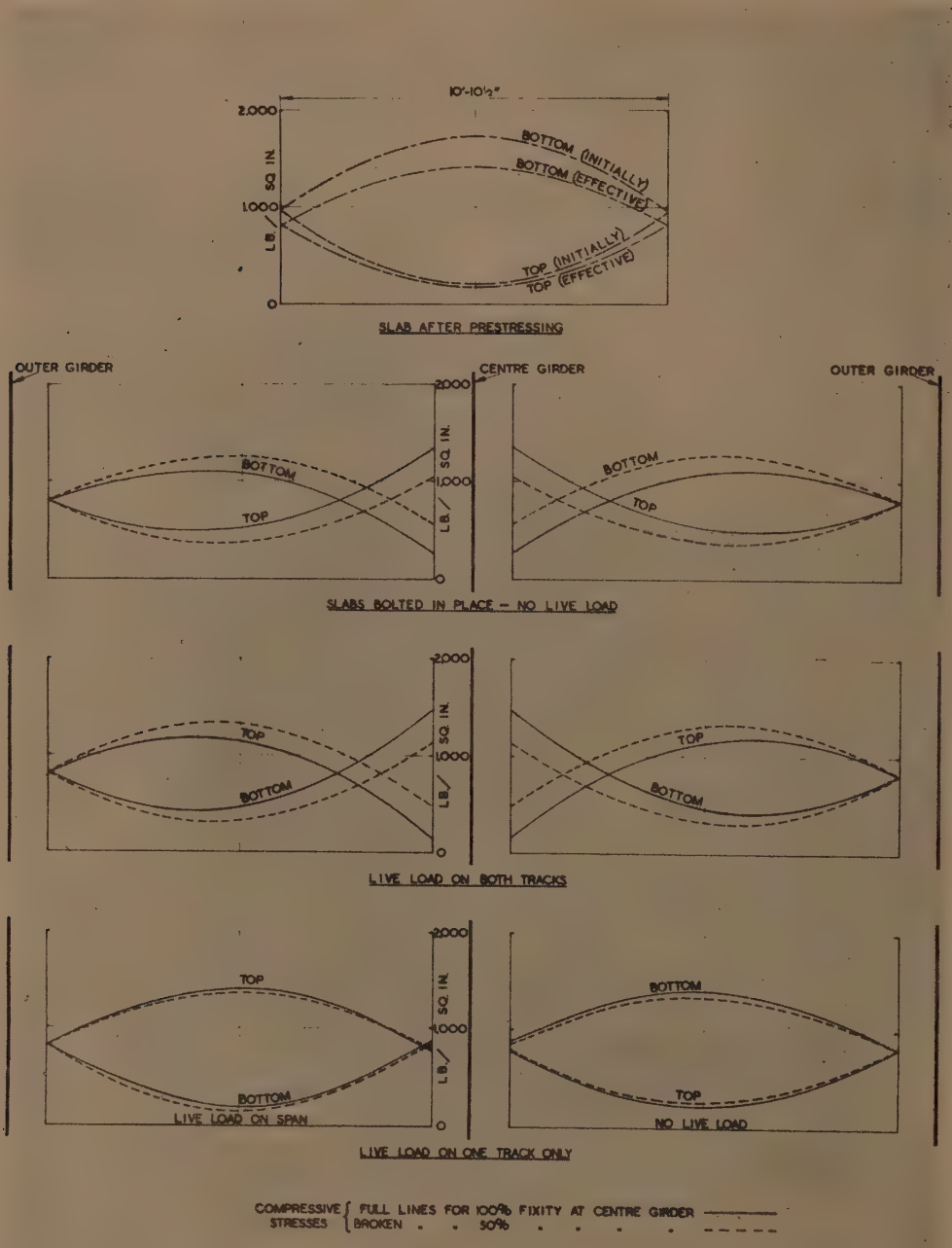
Slide 4 showed one of the units being prestressed. A dial gauge at the centre of the unit was used to measure the hogging ; it amounted to 1/20th in.

Slide 5 showed the end of a unit.

On page 310 of the paper, he continued, Mr. Lee had referred to the change of steel stress in a bar initially tensioned to 40 tons per sq. in., and to the loss of pre- stress by shrinkage and creep of the concrete amounting to about 16 per cent. ; Mr. Lee had stated that that lowered the tension in the steel before the bridge was brought into use by some 3 or 4 tons per sq. in. A loss of 16 per cent. would mean a lowering of that tension by 7 tons per sq. in. ultimately. Mr. Berridge wondered if the 3 or 4 tons per sq. in. referred to by Mr. Lee was intended to apply to some stage before all creep had

finished. He asked for clarification of that point and for some indication of the time required for all creep to stop.

Mr. LEE said Mr. Berridge had raised a number of points rather too fast for him to take them all in and to think of the right answers. He did not know the 150 ft. span girder at Monmouth, but he did know



Slide 3

about the 160 ft. girder at Birmingham. He did not withdraw what he had stated in the paper, but perhaps he had not explained it very well ; and he was not sure Mr. Berridge was suggesting that what was stated there was wrong. The great advantage of prestressing an existing girder was that one added some high tensile steel and prestressed it, thereby taking stress off the previously existing steel. As Mr. Berridge had shown, the strength was increased appreciably merely by a little welding of some cleats, adding some high-tensile bars and using prestressing jacks for a day, whereas if the job were done in the usual way one might need to obstruct one railway track by staging and shoring, which would be very expensive. If one strengthened a lattice girder by adding the equivalent amount of load-carrying capacity as would be obtained by high-tensile bars but in the form of mild steel plates, then one would have less deflection under live load than with the prestressed girder. That made no difference because as Mr. Berridge had said, the deflection was much reduced compared with what it was before strengthening it was cheap, and the method did not interrupt either rail or road traffic.

The other statement in the paper, added Mr. Lee, was also correct, that if in a new lattice girder one under designed the tension boom because it was going to l

estressed, one would have greater deflections than uld occur in the corresponding fully designed mild el lattice girder. He agreed entirely with Mr. rridge's remarks.

On the question of prestress, he believed that in the se of the Monmouth girder the stress applied in sion was 27 tons per sq. in.

Mr. Berridge agreed.

Mr. Lee said the normal initial stress with prestressed ncrete using these bars was 42 tons per sq. in. He reed entirely with Mr. Berridge because, as was well

Mr. L. R. Creasy (Ministry of Works) complimented Mr. Lee on having presented a very valuable record of problems posed and solutions found. It was all the more valuable and all the more welcome since it was concerned with a field of construction where perhaps there had been a somewhat over-emphasis on the laboratory and the philosophical aspects, rather than on the opportunities afforded for ingenuity in the solution of many of our current problems.

The paper, posed as it was, as a case-book of works actually constructed, invited comment in a similar vein,

Slide 4

own, an alternating stress (fatigue to failure) graph any steel was in the form of a cone and when pre-essing steel it was desirable to use a somewhat lower ess as there were alterations of stress at the end chorages instead of none in the case of bonded pre-essed concrete.

In a prestressed concrete beam with an initial stress tons per sq. in., there could be three million altera-ns of double the normal variation in steel stress thout fatigue failure, as no test which had been rried that far had produced failure.

In short spans the grouting and the bond stress evented variations of stress being carried to the ends. hen strengthening an existing lattice girder, it was necessary to be economical with the high-tensile el, because prestressing was very inexpensive and e more bar made very little difference to the cost.

If he had mentioned a loss of only 3 or 4 tons per in., this was only applicable to cases, as in some types bridge, where the concrete had more time for shrink d harden before final stressing.

and Mr. Creasy hoped the meeting would be interested in some other illustrations of works executed which he put forward by way of comment on some of the points Mr. Lee had raised.

Figs. 11 and 12 in the paper illustrated the stressing of a prestressed concrete hose drying column ; an example of quite an economical combination of pre-stressing and normal cast *in situ* construction.

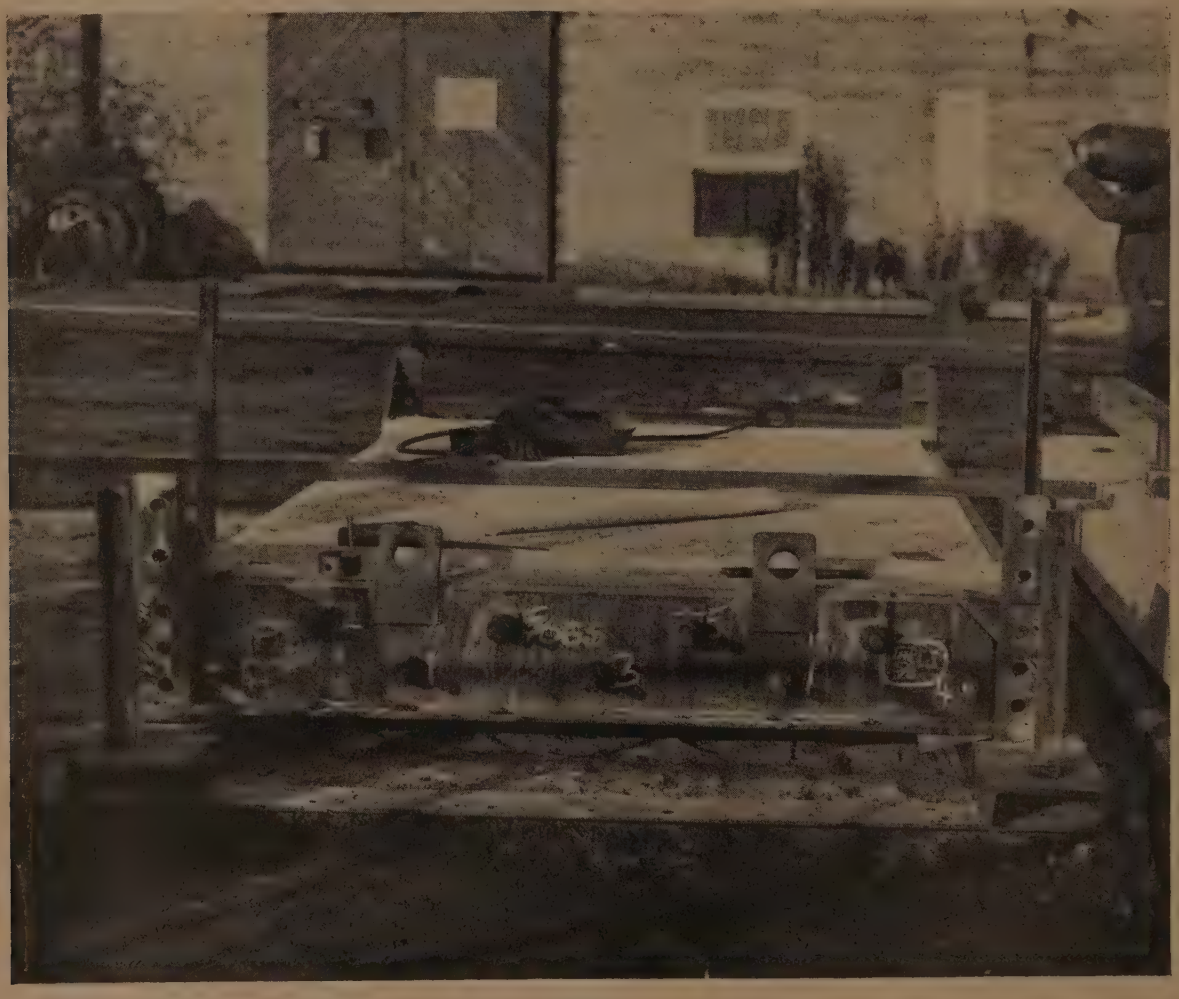
Slide 1 pictured the column as finally erected and in use. It was 40 ft. high, tapered in octagonal cross-section from 16 to 14 in., and was stressed with four high-tensile steel bars. The bars were stressed in pairs in two stages to half and final stress ; the holes through the column for the bars were cored with Ductube, and the ends of the column were lightly reinforced. After careful consideration of alternative methods it had been found that the most economical capping method was to erect the column and then to provide scaffolding to the top from which to cast the cap *in situ* starter bars projecting from the end of the shaft for this purpose.

In commenting on the author's reference, on page 305, to a span to depth ratio of 30 for roof and floor construction, Mr. Creasy showed in *Slide* 2 an example of prestressed hollow concrete units used over considerably greater spans, in that case 35 ft. The units were 8½ in. deep and the span to depth ratio was of the order of 50. There was 2 in. of light-weight insulating concrete as topping, and that had reduced the span to depth ratio to something like 40. It was interesting to note that some of the units, as a result of prestressing, arrived with an upward camber of ¾ in. It was fair to say that the prestressed hollow roof had no greater resilience than one of normal hollow concrete construction of normal span.

Slide 3 illustrated that the erection of these slender units was not unduly difficult.

Coming to Mr. Lee's reference to the use of high-tensile bars on the turning vanes of a wind tunnel, Mr. Creasy referred to the difficulty of obtaining a satisfactory panoramic view of such a wind tunnel. *Slide* 4 showed a half view of the tunnel, looking towards the small end. The overall length was about 400 ft. The working section was rectangular in cross-section and of dimensions 13 ft. by 9 ft., whilst the general cross-section of the tunnel was octagonal and of 45 ft. diameter at the largest point.

Slide 5. An internal view of the small end of the tunnel when nearing completion showing the vanes, of aerofoil section, erected in position. The turning section of the tunnel was octagonal in cross-section and about 25 ft. in diameter, the larger end being about 45 ft. high. The turning vanes to the large end were tensioned with four high-tensile bars, and those to the smaller end with two high-tensile bars. A modern lean-mix concrete was used of low water cement ratio heavily vibrated to facilitate rubbing down to a smooth finish when stripped. The vanes were precast on the site in one section between the cast *in-situ* transomes placed at third points in the height of the turning section. The vanes were stressed after they had been erected in position and their ends cast with the transomes.



Slide 5

In order not to disturb the main shell during the process of prestressing, the outer vanes and margin and likewise the upper sides of the octagon at that point were not cast until after the vanes had been stressed.

Slide 6. A general view of the casting yard. The vanes were cast in one between transomes to a maximum of 15 ft. high, with polished mild steel moulds for the large vanes and teak moulds for the smaller ; and with a light mesh reinforcement. The holes for the pre-stressing bars were cored with 1¼ in. diameter rods afterwards withdrawn. *Slide* 7 showed the process of stressing the vanes after they had been put into place prior to the casting of this section of the upper face of the tunnel shell and showed the shuttering in between the vanes for this latter process. The completed section of the tunnel which was shown in the distance would line up with these upper faces after the stressing of the vanes was completed.

In conclusion, Mr. Creasy again thanked Mr. Lee for his most practical paper.

Mr. LEE said that theoretically the natural frequency was getting near to the frequency of likely repetitive

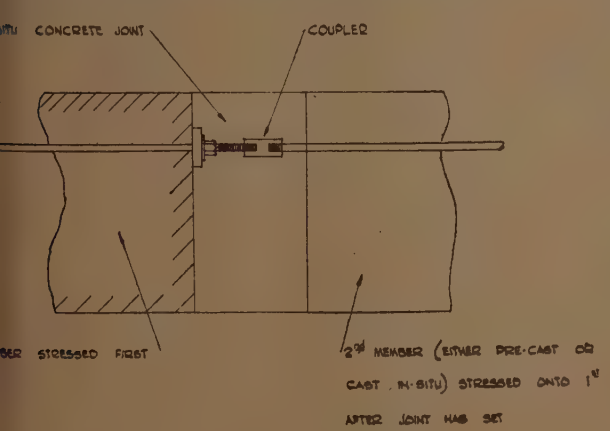
ıding when the span to depth ratio reached 30 or 32. ıat was the position if we ignored damping, but there ere possibly cases in which damping was very effective. ı the other hand, he did not think we had enough idence to be able to say that damping could be relied on to any extent in the case of floors where repetitive

Slide 2

e loading might be applied near to the natural fre- ıency. He agreed, however, that Mr. Creasy was ght in more or less ignoring that in the case of a roof.

In the case of floors, however, a not-too-shallow nstruction, say with ribs at 15 in. to 30 in. apart, was ually cheapest on long spans, and certainly saved eel.

Judging by the dimensions of the wind tunnel referred , which he had never seen, it was much more impressive an the photographs were able to convey.



Sketch A

Discussing the erection of a structure where a number precast members needed to be joined together, he ıstrated a method whereby one could stress the bars by eans of a jack and could use couplers to stress on to that ember another member either being poured-in-place precast. The second member, however, was normally stressed only after connecting up to the first member.

Mr. O. J. Masterman (Associate-Member), was glad of the opportunity to welcome the paper, for a reason which had nothing to do with the fact that he had played a small part in some of the early developments of the method described therein. His reason was that we now had a complete system for prestressing which was originated by a British Consulting Engineer in co-operation with a British firm of steel manufacturers. If we considered the well-known stressing systems which had come from abroad, we had from France Freyssinet flat jack which required no cables or wires at all ; Professor Magnel's sandwich plates from Belgium; the bonded method used most broadly perhaps by Hoyer in Germany and which was taken up by Bjuggren later in Sweden ; then the interesting method of stressing large circular tanks, which had come from America ; and the cunning method of Leonhardt, in Germany, involving the use of a concrete jack which was left in to become part of the structure.

In all cases except the last one we had imported the systems from abroad. It was very pleasing, therefore, that we had now a British system, which enabled us to reverse that state of affairs and to sell abroad in a big way. That change had occurred in a very short time after the development work was done. It involved the use of rods which were very simple to handle and which could be tensioned quickly, and it had the advantage over all the others that there was no anxiety with regard to slip.

Professor A. D. Ross said that he, also, like Mr. Masterman, felt the liveliest pleasure at the development of a British system of prestressed concrete. He believed that, until now, many British structural engineers had suffered a sense of inferiority when they observed the works constructed in prestressed concrete abroad, particularly in France and in Belgium. The paper;

however, would go a long way to correcting any such feeling of inferiority.

Endorsing the remarks made concerning the positive anchorage with this system, Professor Ross said he had had the privilege of supervising some early tests and had had the opportunity to measure strains in steel during stressing and anchoring. Instead of there being a loss, it was even possible to get a slight increase of stress, and one could say with certainty that there should never be a loss on anchoring. That was a great comfort not only to the designer but also to the man on the job.

A feature of the system, which might on occasion be of advantage, was the possibility of restressing at a later date to recover the early loss of stress. If one wished particularly to minimise the loss one could postpone grouting for a time and then restress, thi operation being very simple because of the convenien form of coupling employed in conjunction with the jack

He could also confirm the negligible creep of th steel. Other advantages, such as reduced danger c corrosion, would be obvious to all structural engineer

Professor Ross was glad that Mr. Lee had draw attention to the possible danger of resonance in beam under forced vibrations, and said that just bef coming to the meeting he had calculated approximat the frequency of one of the early test beams of 20 span. Under self-weight alone, the frequency h worked out at 80 vibrations per minute ; then he h assumed that the total load was three times the s weight and that gave a figure of something under Whilst he was not an expert on dancing, eurythm

Slide 3

Slide 4

the like, he suspected that these recreations might use periodic disturbing forces with frequencies somewhere within this range. It was particularly important prestressed concrete since the damping was not very arked. In other words, the position might be potenally dangerous, and he considered Mr. Lee was wise be just a little cautious about the matter.

He had been very interested in the tie-bars for portal ames and arches proposed by Mr. Lee. This was an genious device whereby we could provide a tie-bar, d within quite wide limits we could control its strain adjusting the area of concrete. He asked whether r. Lee would confirm that, provided corrosion was no fficulty, a larger quantity of mild steel might be econoically cheaper. But by using high-tensile steel bars orking at reasonably high stresses we could put the oncrete to a dual purpose, firstly to control the extension the tie-bar, and secondly, to protect the steel, which ould generally be in the rather unfavourable situation being in the ground.

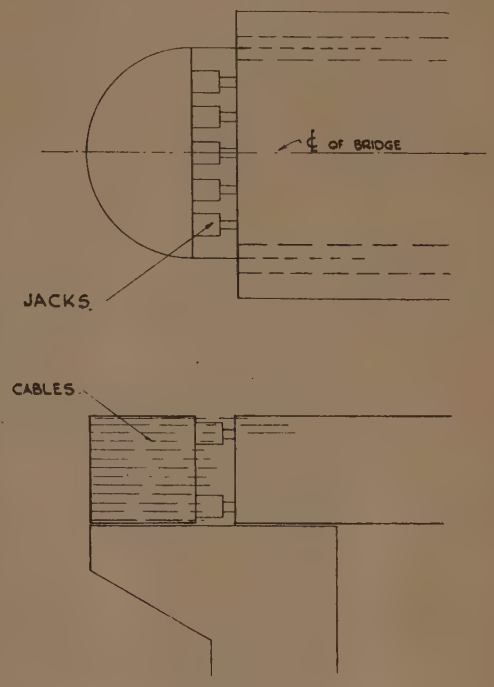
Slide 5

The strengthening of bridges was most interesting nd seemed to him to offer a splendid application of the lloy steel bars. He had only speculated on the possiility of applying the method, not only to simple atically determinate frames but also to statically ndeterminate structures. He was thinking of the ultiple braced girder, in which we might obtain a eduction in dead load stresses in the upper boom, rovided the web members could, without overstressing, ansmit the shortening effect from the bottom boom. oubtless there were also useful fields of application in he strengthening of continuous and other redundant ructures.

Finally, Professor Ross said Mr. Lee had given the sults of what had been a very long and arduous piece development work. In the paper he had shown the end product," and those who had undertaken developent work would appreciate the vast amount of time nd energy which had gone into the work described.

He expressed gratitude for the opportunity afforded him to attend the meeting and the presentation of a most interesting and important paper.

Mr. LEE, in a comment on Mr. Masterman's remarks, said the Leonhardt system was one used for bridges, mostly in Germany and he knew of one also in Switzerland. Illustrating the method used, he said that very powerful jacks were used as the whole bridge was



Sketch B

Slide 6

stressed in one operation and the jacks were preferably all operated by one large electrically driven hydraulic pump.

Referring to the positive anchorage, mentioned by Professor Ross, he agreed that in stressing it was possible, especially by means of the new nut tightening device, which had a hand-operated worm drive, to apply a little more stress to the bar ; if the gauge read 43 tons, one could put on another 3 by tightening the nuts.

The creep tests made by Professor Ross had been most valuable, and had subsequently been repeated in America, with close agreement.

Slide 7

As regards the ability to restress in the case of the River Don railway bridge, it had been useful to stress 30 or more bars to verify any friction loss and then to stress finally afterwards.

With regard to tie-bars, in the case of the larger hangar at Meir airfield, where there was a structural steel portal frame of 150 ft. span, he had used mild steel for the tie, but if he did it again to-day the high-tensile steel would only have needed to be one-quarter the weight and the end connections would have been easier. As Professor Ross had said, tie-bars were generally in unfavourable locations and so concrete casing was usually needed anyway.

Mr. A. HILEY (Retired Member), said that on reading the paper he was certain that he would find occasion to learn something of up-to-date interest ; he asked Mr. Lee for enlightenment on some matters about which he was not quite sure, touching the high efficiency couplers and nuts used for high-tensile steel bars of Figs. 15 and 16.

In THE FINANCIAL TIMES of December 10th, 1952, there was published a very good review of British overseas civil engineering contracts throughout the world, the value of which amounted to around £62 millions, and in that article there was a reference to the Tampa Bay bridge. The materials we had exported for use in that bridge had brought into this country 125,000 dollars, as given in a tabulated list, or about £44,000. He asked Mr. Lee whether that figure represented the cost of the " Macalloy " steel exported for such use, for he felt the figure was small as compared with the value of contract in Section A and B, for which alone the paper mentions 144,000 dollars, was an estimated nett saving attributable to prestressing.

With regard to the tightening of the bars and the slight loss of tension which occurred some time after a structure had been completed, he wondered whether the American method of fitting the bars within a medium inside the concrete would allow them to pull through slightly, to facilitate the recovery of lost strain when found desirable, as distinct from bars which were grouted and bonded to the concrete throughout their length. If that were so, he asked whether the use of couplings associated with a suitable means to lengthen the tie rods so that further tensile stress could be applied at their ends and be resorted to after the completion of a bridge or an individual pile. The designer might thereupon be able to avoid even the loss of the allowable 16 per cent. or so of tension.

It had occurred to him, in regard to tightening tie bars, that it might be better to put the nut for the tie-bar on a broadened plate if readjustment of tension had to be undertaken. If post-tensioning were required, it might be possible to extend the coupler by giving a differential-screw effect, so that we could use a thread pitch of virtually one-tenth the normal amount, which meant that the exerted stress could be correspondingly increased. He threw out the idea because he had made something on those lines on one occasion, and it had worked well as applied to tubes.

Mr. Hiley recalled that in 1931 Mr. C. J. Jackaman had presented a paper on the piling of Ryde Pier. Many of the piles there had sunk 6 or 8 in. further than was intended, and difficulties had come about ; the question had arisen as to whether extension in depth could be provided for a pile column to reach depths which were not possible by normal piling. Mr. Hiley ventured to suggest that, with the advent of better methods entailing prestressing, we should be in the position some day to cope with such situations and possibly make piles twice the length which the piling frame would of itself permit.

A pile of 13 in. section in ordinary concrete was stated in the paper to be equivalent to a prestressed pile of 10 in. section, presumably with a uniform central bar. That implied that the forces and bending moments exerted were very much eased, so that the piles might be driven by lighter gear, or alternatively be driven more effectively by the normal gear. He felt that the gain in strength due to prestressing was an important advantage because a considerably longer length of pile could thereupon be driven and handled with enhanced safety.

He supposed such a pile if driven within a tube or outer casing would be of octagonal section, and he asked whether the centre rod, without necessarily being solid, could be tubular if made of " Macalloy," so that some cement or chemical agent could be injected into the ground if required.

Some composite piles with which he had been associated were formed with a 16 in. preparatory tube and were cast *in situ*, such tubes could be driven for a depth of (say) 50 ft., and he had for the special purpose devised arrangements to close temporarily the ends of the tubes during driving. If a greater depth were required than the said 50 ft., it was necessary only to introduce another pile of 12 in. octagonal section into the guide-tube when it was fully driven into the ground.

n such a manner composite piles reaching 70 to 80 ft. in epth were installed at Battersea Power Station in 1930. Referring to the prestressed tubular pile illustrated n Fig. 16, he asked whether Mr. Lee would, given the neans, make the core with a removable pneumatic tube. n that connection he recalled that in America in 1932 "Flexicore" tube was constructed for this use in ccordance with U.S.A. Patent No. 1949650 and introuced to form the 10 in. core, being deflated afterwards or withdrawal. It had given very good results in Kansas where a 24 in. octagonal reinforced concrete ile 40 ft. in length, driven to 200 ton resistance, safely ook a test load of 100 tons.

It was curious, he commented in conclusion, that owadays we were resuscitating some of these older deas. and could doubtless combine them to much dvantage in future, thanks to improvements in techiques involved with prestressed concrete for instance uch as were aptly described in the paper.

Mr. WALLACE EVANS (Member), speaking of long-span ollow floors, asked for Mr. Lee's view on what would e the effect of making them continuous and thereby chieving some degree of damping. Arising out of that, e asked whether, with the system described, any rogress had been made with continuous beams of any ind.

Secondly, he said that in the Tampa Bay bridge the tiffeners had one bar through the centre, and he wonlered whether that was by design. In that connection e mentioned a bridge of slightly longer span, but of imilar section, where it was convenient to stress the eams in two parts, first to carry the dead load of the tructure, and secondly, when the beams were in position, o stress them again to take the live load. The result vas that the stressing of one beam affected all the other eams in the bridge, showing that the stiffeners transnitted the stressing to all beams. Loading tests were arried out in order to determine whether this applied to he load also, and it did. He suggested that because f this it would be better to have two bars, one at the op and one at the bottom.

Mr. LEE, commenting on Mr. Hiley's reference to the rticle in THE FINANCIAL TIMES, said he was not conerned with the sale of steel and he was not able to nswer the question.

The curve for creep was asymptotic, of course, and with post-tensioning the concrete had at least half its 28 day strength before stressing; much of the creep occurred in the first week but it was not often worthwhile to restress before grouting, although very easy to do.

The loss for creep and shrinkage was about 16 per cent., and by restressing one could halve that loss. But there was not much point in doing so, because the loss of prestress was not exactly a real loss since it did not reduce the ultimate strength. In the case of the Sheffield bridge, which was cast in sections, one could avoid nearly all the loss through shrinkage of the concrete and reduce the loss due to creep by most of the concrete having ample time before the stressing.

He hoped he had satisfactorily answered Mr. Hiley's question concerning slip rods and restressing to reduce losses.

Of course, the driving of piles 160 ft. long with a pile frame 80 ft. high, was not new. Mr. Hiley would be the first to say that one could add extra length to a steel pile by welding. With prestressed concrete there was not much difference, but the joining of the sections together was more tricky.

The question as to whether a 13 in. section ordinary concrete pile was equalled by a prestressed concrete 10 in. section pile was one to which several answers could be given; it would take more time than was available to him to answer it.

The driving of a hollow pile through which another pile could be driven, the hole afterwards being filled with concrete, was an interesting method, but he did not think it was often economical, because there was a very big hole to fill.

Replying to Mr. Wallace Evans, Mr. Lee said that the axial transverse prestress of the Tampa Bridge design was in his opinion adequate and he had no doubt all the beams acted together as in tests of other bridges but he did agree there might be some cases where top and bottom bars were desirable. Mr. Wallace Evans had perhaps noticed he had used them in the case of Leyton Marshes. With the recent rapid developments and experience both in construction and design Mr. Lee thought he would not be likely to repeat exactly the Tampa design although it was only two years ago that it was done.

The Construction of Eight Prestressed Concrete Tanks*

Discussion of the Paper by Colonel A. R. Mais, O.B.E., T.D. and Mr. A. C. Little

The CHAIRMAN welcomed the members of the Reinforced Concrete Association and their President, Mr. M. Cowlishaw, and then introduced Colonel A. R. Mais, O.B.E., T.D., and Mr. A. C. Little, who presented their Paper.

Discussion

Mr. M. COWLISHAW, President of the Reinforced Concrete Association, thanked the Institution for the welcome they had given to members of his Association and also for the frequent use which the Association made of the Institution's Hall. He then moved a hearty vote of thanks to the authors for their excellent Paper.

Lt.-Col. G. W. KIRKLAND, M.B.E. (Member of Council), who opened the discussion, referred to the first sentence of the Paper, where it was stated that the Ministry of Works had invited competitive tenders and designs. This, he felt, was a regrettable waste for a Government Department to require five or six designs for a job which could be done admirably by one consultant. Much of the labour employed by this method was completely wasted and while not detracting from the excellence of the completed tanks, there was no assurance that the client was better off as a result.

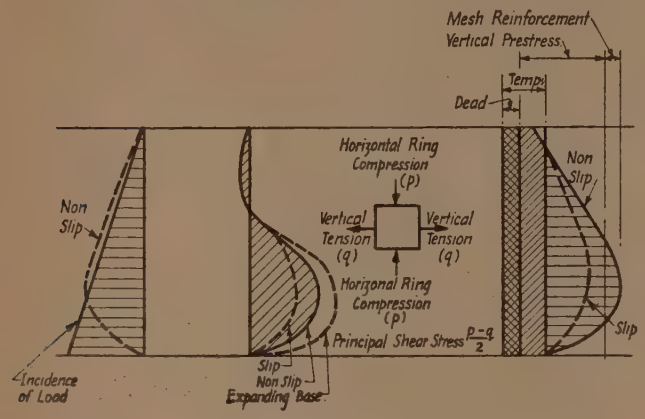
Secondly, Colonel Kirkland commented on the laudability of the selection of prestressed concrete for

*Paper read before a Joint Meeting of the Institution of Structural Engineers and the Reinforced Concrete Association at 11, Upper Belgrave Street, London, S.W.1, on Thursday, January 8th, 1953. Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President of the Institution of Structural Engineers), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 1, pp. 1-8 (Jan., 1953).

the work, especially since hoop tensioning had been proven in the reinforcement of gun barrels in the days long ago when the cannon went into disuse.

One point about the design which had struck him rather forcibly was the considerable heaviness at the base of the structure. Did the designers consider the use of an inverted saucer type of base before they made their choice? The tensions would in that case be provided purely and simply on the periphery.

Finally, one word of regret and severe criticism : why use rolled steel joists in the roof? Could not these have been prefabricated and post-tensioned in concrete? Their shape lent them admirably for this ; they would have been quite economical with the large number



Vertical M's
Slide 1

required ; and they could have been cheaper than the steel joists. A completely prestressed tank would have resulted.

Colonel Kirkland expressed his pleasure at having had an opportunity to hear the Paper and congratulated both authors on its presentation.

Mr. L. R. Creasy (Superintending Structural Engineer, Ministry of Works) said that when comparing prestressed concrete tanks with those of more conventional type, the attraction from the designer's point of view was that a condition of maximum stress was achieved when the tanks were empty and was gradually relieved when the tanks were filled. In the circumstances, it was surprising that, in this country, we had such relatively small experience of prestressed concrete tank construction.

Thanks were therefore all the more due to the authors for their excellent exposition of some of the difficulties which were to be experienced in construction of this type.

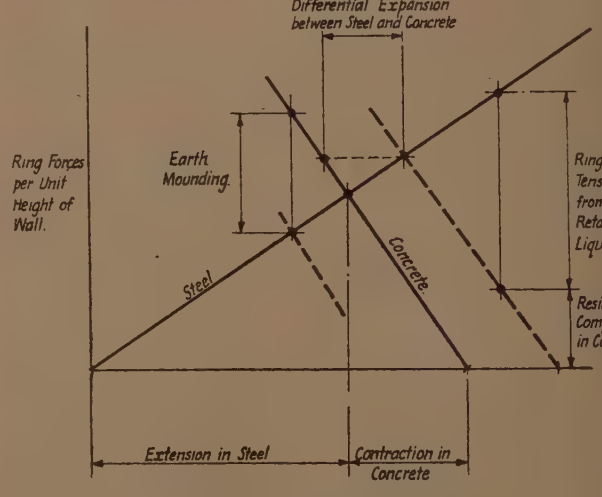
Mr. Creasy said he wanted to demonstrate, by slides and diagrams, something of other ways of dealing with prestressed concrete tanks, and to refer to one or two questions on design which had not been so far mentioned.

The authors had referred to the difficulties to be encountered with a fixed wall-to-base-slab form of design, but it ought not to be assumed that if the joint were sliding, all the difficulties were easily resolved. The first diagram, on the left of the first slide, showed the ideal to be aimed at whereby the induced tensioning in the vertical section of the wall was of triangular form equal and opposite throughout the depth of the tanks to the induced ring tension exerted by the liquid. Where the base was pinned, there was a falling off of the incidence of the load, as shown by the dotted line, and in practice, where the base was free to slide—if it could—there must always be an uncertainty as to what intermediate position would be achieved between complete slip and a pinned condition.

The next diagram in the same slide showed moments in the vertical section of the wall, induced first of all by tensioning and then relieved by moments of similar form by the water. The full line showed the condition of moment induced with a pinned base of wall, and where complete slip took place, the moment would approach to zero. In practice there would be some intermediate condition achieved where the wall foot tended to slide to an indeterminate extent. If hot liquids were to be stored and the tank base tended to expand, the wall foot would be pushed outwards with increased vertical moment as shown.

The diagram on the right of the same slide indicated the build-up of extreme fibre stress. First of all, the internal fibres would be prestressed throughout by the dead weight of the concrete construction. With an unroofed tank, this would tend to be of triangular form and with a supported roof it would approach the rectangular form—the weight of the walls being relatively minor in effect. If the liquid were heated, there would be a tendency for constant compression on the inner fibres and from solar heat on the outside the internal skin would be in tension. The extreme fibre stress from the wall tensioning must be added as shown by the full line representing the " pinned " condition, and this would fall away as shown by the dotted line if it were assumed that some degree of slip had taken place. The vertical prestress would be so proportioned as to offset the algebraic sum of this extreme fibre stress, and remembering that this was the condition before the tank was filled, it appeared reasonable to tension the wall for something less than would provide for this most extreme condition.

The small diagram was a reminder of a further complexity which occurred in this type of problem. The horizontal wing compression induced by the tensioning of the wire on the outside was shown in conjunction with a small vertical tension. It was common to provide a vertical compression to overcome all induced tension stresses due to loading conditions. But even

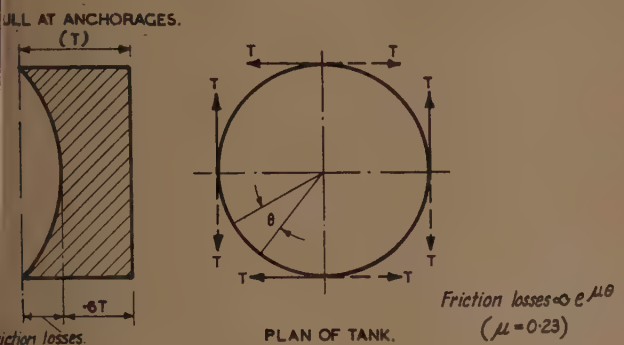


Circumferential Deflections
Slide 2

apart from the small residue of tension and bearing : mind the familiar formula, the principal shear stre would be of the order of one-half the induced compressio so that when working to a ring compression stress 1,000 p.p.s.i., there would be an induced principal she stress of the order of 500 p.p.s.i.

The second slide dealt with some of the complexiti which arose in dealing with horizontal tension wh computing the extension of the steel and the induce

formations in the concrete. Mr. Creasy was indebted Mr. Hancock, one of their senior engineers, for the agram which neatly illustrated the possible alter-tive conditions. Ring forces per unit height of wall ere shown vertically—forces due first of all to tension-g externally and secondly to the entry of the liquid. xtension in the steel was measured horizontally from it to right and contraction in the concrete from right left. Thus the vertical and horizontal ordinates om the line marked " steel " going upwards to the ght of the diagram, represented respectively the ring



Horizontal Tensioning
Slide 3

nsion and the corresponding extension in the steel. he vertical and horizontal ordinates to the line from ght to left marked " Concrete " represented respect-ely the ring compression and the contraction of the ncrete. For equilibrium a common position of equal d opposite ring forces would be met on both graphs hich could be adjusted as necessary for any required agnitude of ring tension from the liquid or residual mpression in the concrete. Likewise, it followed that rizontal ordinates to the concrete line measured from e position of equilibrium to the right would show lief of contraction or expansion in the concrete from e stressed condition.

The diagram could also be used to illustrate, graph-ally, the various alternative of loading. Supposing e tank were mounded, thereby introducing an addi-onal compression in the concrete, with corresponding lief of stress in the steel. Movements to the left from e original point of equilibrium in the diagram repre-nted relief of stress in the steel, with corresponding creased contraction in the concrete so that after mputing the ring tension from the mounding the new sition of equilibrium could be established in the anner indicated.

Supposing the tank was subject to solar heat or the ater in the tank was heated, with consequent differen-l extensions between the steel and the concrete. om the original position of equilibrium the extension the steel was represented by movements to the right ong the " steel " line and likewise relief of contraction the concrete would be represented by equal move-nts to the right along the " concrete " line until a w position of equilibrium was reached as indicated.

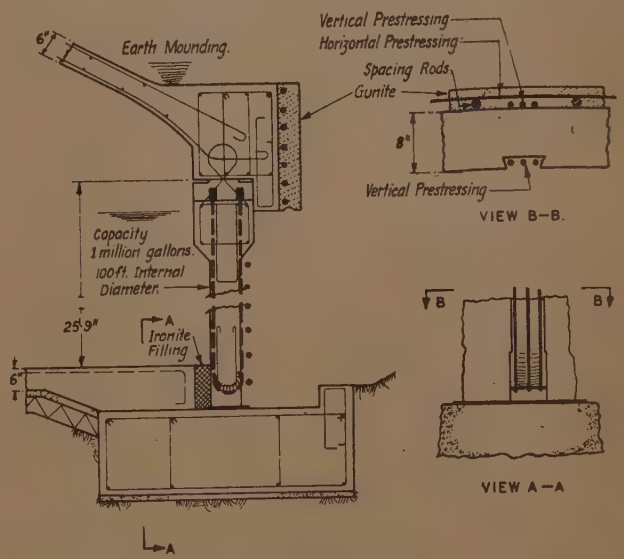
Similarly, in dealing with the induced ring tension om the contained liquids, extensions in the steel and ief of contraction in the concrete were represented movements to the right from the position of equili-brium along the " steel " and " concrete " line respect-ively and are plotted from the logical ring forces in the manner indicated.

Mr. Creasy also demonstrated a question on the friction losses when tensioning horizontally. The third slide illustrated some results of tests which had been made during actual construction to assess the frictional losses encountered in the wire from two-point stressing. They found that the losses followed the familiar exponen-tial formula in which the coefficient of friction was of the order of 0.23 from which the mean value for the effective pull in the wire would be of the order of .8 of the pull at the jacking points. The wires in this case were protected by a plastic sheathing.

The authors had referred to the difficulty in casting the vertical stressing wires in the thickness of the walls, and to the scaffolding which that necessitated. Mr. Creasy's fourth slide showed a tank where both the horizontal and vertical tensioning wires were placed and tensioned after the tank had been cast. A chase was cast on the inside face of the wall, with a slot at the base through which the wires were laid, and the wires passed to the end anchorages through slots cast in the top of the wall. In this case, to reduce friction, mild steel rollers were used as shown.

Mr. Creasy said that the base slabs of tanks described by the authors had been directly prestressed by cables in the slab but that he had tried an indirect procedure which would introduce some small residual compression in the base slab of the tank after it had been filled. Three cables were first tensioned at the foot of the tank wall with the object of producing positive and calculable slip inwards. The joint at the base was then filled with ironite, used in this case because the liquid was deemed to attack a normal bitumastic filling.

When the ironite was set the remaining wall cables were tensioned which produced a further inward move-ment of the tank wall and compressed the ironite filling between base slab and vertical walls. Finally, three cables were wrapped on to the base of the wall to increase

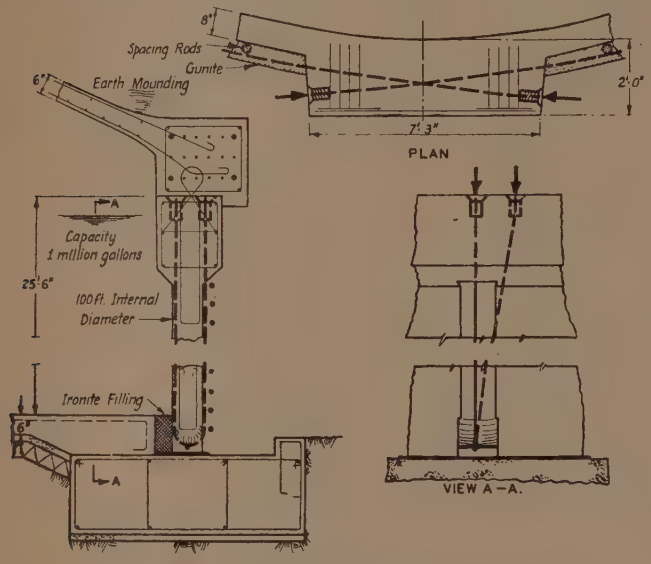


Wedge Anchorages
Slide 4

further the prestress in the base slab. When the tank was filled with liquid there was a small residual com-pression across the slab and joint at the wall foot. It would be noted that the detail of the external rim to to the base slab formed a shoulder to retain additional external sealing to the base joint and provided for the " belt and braces " mentioned by the authors.

Dealing with the form of anchorage to be used, Mr. Creasy said that the authors had used wedges, for which a neat double acting jack was produced which enabled the wires to be tensioned in opposition in one operation. That was very neat when dealing with horizontal tensioning but the same simplicity was not produced vertically since the group of tensioning wires in any one slot must be pulled two at a time.

The fifth slide dealt with cone anchorages which possessed the advantage of enabling a group of wires to



Cone Anchorages
Slide 5

be pulled in one operation but were less efficient horizontally. The jacks in this case were not double-acting and did not enable the wires to be pulled in opposition so that a massive pier was required in the wall as a seating to the jacks. It had been found that when tensioning horizontally there was a tendency for the cables to cockle. Experiments were being tried with special jacks to overcome the necessity for the construction of the pier.

The sixth showed a tank in which wedge anchorages had been used for the horizontal wires and cone anchorages for the vertical wires and showed the alternative of looping the vertical wires cast in the wall. These latter were wrapped in a plastic sheath, and not grouted, which overcame many of the difficulties with steel tube core for the vertical wires as mentioned in the Paper. Horizontally, the wedge system was used with mild steel bar rollers to reduce the friction losses when tensioning horizontally.

The seventh slide was a general view of one of the tanks, showing the cabling coming up into the top of the wall. It showed rather a large expanse of base slab cast between construction joints which had caused trouble and was later reduced.

The eighth slide showed the pier on one of the tanks using cone anchorages, and the corresponding disadvantages were obvious. The last two slides showed close-ups of the horizontal cables, one with and one without the spring core in which the wires in both methods showed a tendency to cockle one on another.

Mr. Creasy said that he wished to reassure Colonel Kirkland on the criticism which he had made concerning the Ministry's methods of inviting competitive designs from outside authorities and contractors. The Ministry of Works did not normally use such methods. It might have been that three years ago they were under the mistaken impression that there were authorities outside the Ministry who knew all about the design of pre stressed concrete tanks and perhaps they now knew better.

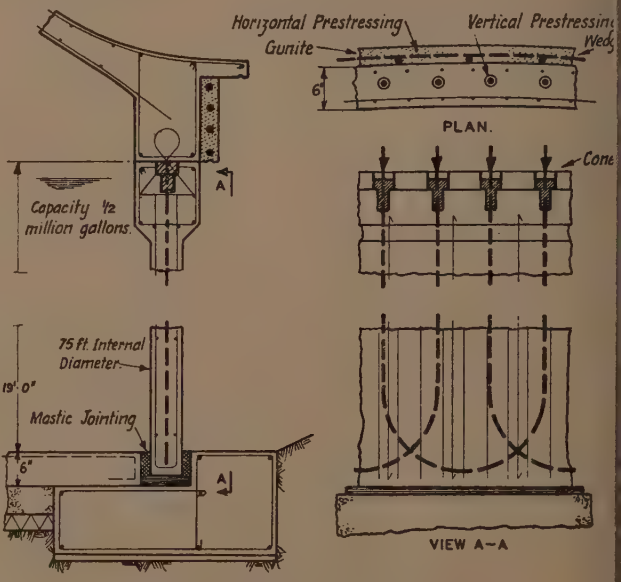
Mr. E. H. Bate said he had watched the tanks under construction and it had been interesting, during the discussion, to hear how the contractors had overcome their difficulties. He was impressed by the fact that where construction joints were left they had leaks, and where no construction joints were left they had no leaks. Speaking of jetting, he said it had been stated that on the horizontal joints a considerable amount of material was removed ; he understood that one of the clauses demanded that two inches should be removed from the surface before the new concrete was placed.

On the vertical joints that could not be done, but the surfaces were carefully prepared. The leakage which occurred was probably due to shrinkage. That raised the question of whether it was worth while using the very rich concretes for water-retaining structures or pre stressed concrete. The concrete in this case was $1 - 1\frac{1}{2} - 3$.

Such a mix when vibrated produced a layer of fine material at the top of each lift, which was the reason for the clause in the specification to which he had referred.

It was possible to obtain the strength required for prestressed concrete with mixes of six or seven to one provided they were properly designed, and with such mixes there was less shrinkage and no layer of fine material was formed so that it was unnecessary to remove so much material. The surface could be prepared by jetting only.

It was worth noting that with properly vibrated concrete stop ends could be stripped almost at once so that the labour of preparing joints was much reduced. They could be jetted down at once and the damage sometimes caused by hammering or the use of power tools avoided.



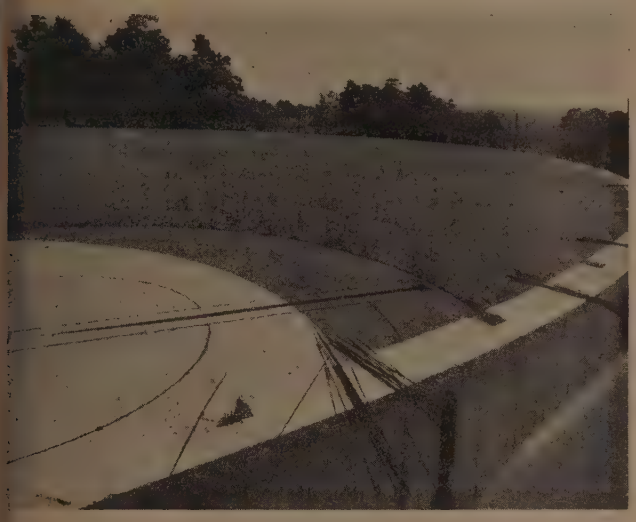
Wedge and Cone Anchorages
Slide 6

Professor G. Magnel (Member) said he did not know when he arrived in London, that the meeting was to take place, and it was a pleasant surprise for him. An astonishing point arose about friction. A few years ago they thought that friction would be less the smoother the surface, but in the end they found the opposite to be the case. Now, in making tanks, they tried to make the surface as rough as possible. The only explanation

he could offer was that with a rough surface the wire touched only at a few points instead of making continuous contact.

One of the speakers had given a frictional coefficient of .25. Professor Magnel had found .3 with a very smooth surface, so .25 was probably the figure to adopt for rough surfaces.

A central cable was awkward for vertical prestressing, but, in tanks where the vertical shaft was fixed to the foundation, they could make the vertical prestressing



Slide 7

by looping the wires in the foundation itself. The wires could be short. The wires for the vertical cables could be joined to the short bits and spliced with several devices which exist ; one way was the use of hard metal splices wherein the wires were fixed with small wedges ; another way was to take two wires, passing one over the other along a certain length and wrapping them around with the Swiss machine, which gave a very satisfactory and cheap joint.

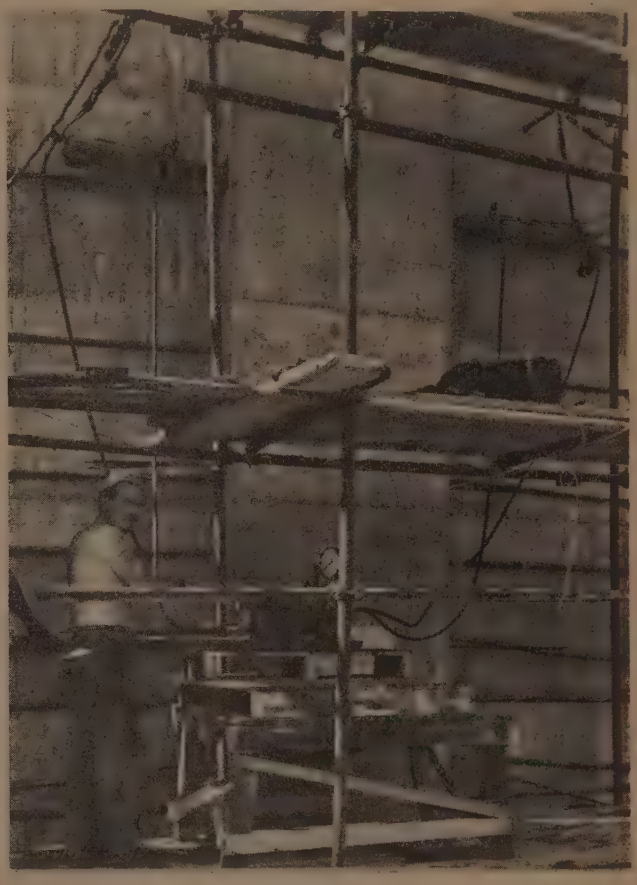
Speaking of cost, Professor Magnel said they must try to make their buildings as cheap as possible, but the first essential for engineers was to make their work technically as near perfect as they could. The prestressed concrete tanks would be much more durable than anything in conventional reinforced concrete, so that, even if the price were a little higher, it was a cheaper job.

Engineers in all countries must strive to make their prestressed concrete technically perfect. There was a good deal of competition between systems, and perhaps some people had expected him to advocate sandwich plates instead of cones, but that was not his idea. They should use the method which fitted best technically with the job ; they should not do things which were ridiculous simply because of a certain system. They should not play with calculations.

Professor Magnel gave an example he had met recently. The Consulting Engineer was limited to about 2,000 p.p.s.i. There were big beams, about 90 feet span. He reasoned as follows : " I will stretch first only 50 per cent. on my wires ; I then wait a couple of days before stressing the other 50 per cent. During those two days a loss of prestress of 20 per cent. will have occurred in the first wires. That means, when the whole cable is stretched, that I have a loss of prestress of 10 per cent. and that the stress in the concrete is only 2,000 p.p.s.i." In fact, said Professor Magnel, if the proper calculations were made the figure was nearer 3,000 p.p.s.i. than to 2,000 p.s.i.

If an accident occurred with prestressed concrete, it would not matter which was the system or the country or whose was the fault ; it would set back prestressed concrete all over the world for years. He appealed to them to be technically honest in all they did.

Mr. J. W. AGER expressed his personal compliments to the authors, as he had himself had some connection with the construction of tanks. They had taken the trouble to ensure that the tanks were built in accordance with



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the best possible practice for concrete construction ; in fact the work was well up to " text-book " standard.

Reply to the Discussion

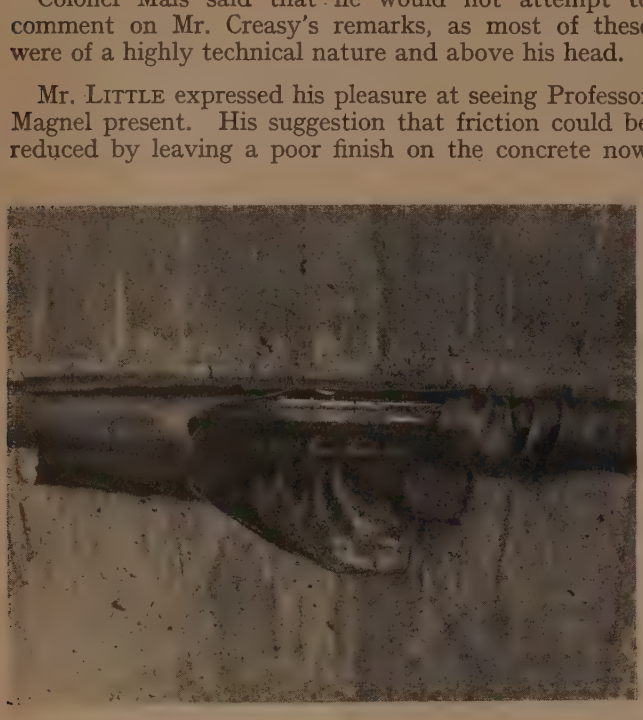
Colonel MAIS said he entirely agreed with Colonel Kirkland's criticism about the roof ; they could have made it a completely prestressed concrete tank and he could only say he was sorry they had not done so. The main reason for not having done so was the need to get on with the job quickly. If they used that type of roof again, which was not very likely, they would certainly go for prestressed concrete.

Replying to Mr. Bate, he said the strength of the mix was an old argument. They had used 1 : 1½ : 3 on the tanks under discussion and at the risk of incurring Mr. Bate's wrath, he would say he intended to use the same mix on the tanks now under construction. The comments on shrinkage were true, but the walls of the tanks in question were not thick and there was no mass of concrete. They had found the weaker mixes to be so unworkable that the cost and difficulties of placing were out of proportion to the saving made by leaving out the cement. They had not yet reached a stage in the technique of placing the leaner mixes of concrete at which they could guarantee a good job on a 9 in. wall.

With a wall of 2 to 3 ft. thick there was a good head of concrete and, under these circumstances, there was a much better chance of achieving satisfactory compaction.

Colonel Mais said that he would not attempt to comment on Mr. Creasy's remarks, as most of these were of a highly technical nature and above his head.

Mr. LITTLE expressed his pleasure at seeing Professor Magnel present. His suggestion that friction could be reduced by leaving a poor finish on the concrete now



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seemed obvious, and it was galling in view of the trouble they had taken. They had flattered themselves on their glassy finish ! Mr. Little said he was struck by the system of splicing prestressing wires and hoped to find out more about it.

He agreed with Professor Magnel's comments on costs and durability, but, human nature being what it was, the struggling contractor had to think about costs. It was not easy to tell a client that a job cost more, even though it was a better job.

Mr. Creasy had shown a system of stressing the wall against the ironite filling. When the tank was loaded there must be some relaxation of tension on the stressing cables and some relaxation in the loading on the wall. It appeared that the pressure of the wall against the ironite filling would decrease, too. Was it not possible to get a leak there ?

The CHAIRMAN, closing the meeting, expressed pleasure that Professor Magnel had taken part in the discussion. No doubt the authors would reply to other points in writing, but the meeting was grateful to them for their Paper and their replies.

Written Discussion

From Mr. E. WINGROVE KEER : Had I been present at the discussion, there is a question I would have liked to have asked regarding the tensioning of the horizontal cables. I am hoping that, perhaps, it is not too late for you to deal with my query by correspondence.

I note from your paper, and from a previous paper by Mr. Ager, that the loss through friction when the horizontal cables are prestressed is considerable, and that although this may be lessened by increasing the number of jacking points, such a procedure is accompanied by a serious rise in labour costs.

From a consideration of this problem, I have wondered whether it has ever been suggested that a possible method of reducing the friction would be to carry the cables on vertical mild steel bars partially embedded in the exterior concrete surface of the tank wall. These bars would be slightly greased where the cables pass over them. They would be spaced as far apart as possible consistent with the diameter of the tank and the thickness of the wall. Thus the cable would be pulled over a number of points presenting very little frictional resistance.

I would suggest that the bars might be $\frac{1}{2}$-in. diameter, half embedded in the concrete, and that the external shuttering would be in plan a straight line between these points. An added advantage would be that the wires would be proud of the concrete by $\frac{1}{4}$-in. all round the tank, thus providing an excellent key for the guniting.

I imagine that the length of these vertical bars could conveniently be made equal to the shuttering lifts, or multiples thereof. The upper end of a bar would project above concreting level and be clipped, also above concreting level, to the formwork. This clip would be disconnected when the forms were struck. The next bar above would be connected to the bar below by a butt joint with a thin metal collar. The bars would be held back to the concrete (if found necessary), by occasional wire ties. Against the additional cost of providing these vertical bars would be offset the saving in the high-tensile cable steel.

I should be interested to know if you think such an arrangement a possibility, both from the point of view of design and construction.

Colonel MAIS replies : The points raised by you with regard to frictional loss on the circumferential cables has caused us all a great deal of thought, and we had, some time ago, decided that a possible solution was the one put forward in your letter, of inserting steel rods vertically in the walls to form bearing points for the

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cables. There is, as far as we can see, no structural difficulty in this method.

I quite agree with your statement that if the wires are to be proud of the face of the tank by means of steel rods or other type of fitting, then it does form an excellent key for the gunite.

In actual fact we had proposed on the next tanks that we construct to incorporate a bar on the lines of your suggestion. If we do finally carry this out I shall be very pleased to let you know our opinion, after we have had practical experience.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, March 26th, 1953, at 5.55 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership?

STUDENTS

BOGLE, John Brian, of Bolton, Lancs.
BRUNT, John David, of Swansea, Glam.
CHALLEN, Maurice Sydney Harold, of London.
HARTLE, George, of Rochdale, Lancs.
HILL, Peter Frederick, of Southend-on-Sea, Essex.
SETTLE, Brian, of Bolton, Lancs.
TERRY, Louis Stanley, of London.
VENIER, John, of London.
WATLING, Terence Roy, of Clapham, Beds.
WILSON, Edward Arthur, of Barking, Essex.

GRADUATES

BALICKI, Joseph George, of Huddersfield, Yorks.
BOND, Rex Edward, B.E., B.Sc., New Zealand, of Wellington, New Zealand.
BULLOCK, Francis Hubert, B.A.(Cantab.), of Leeds,
CLARKSON, Trevor Kenneth, of Stafford.
CONNOR, David Young, of Bishopbriggs, Lanarkshire.
COOK, John George, of Wirral, Cheshire.
FAIRBAIRN, Derek Sidney, B.Sc.(Eng.) London, of London.
FORBES, Bruce Douglas, B.E.(Civil) New Zealand, A.M.I.C.E., of Christchurch, New Zealand.
HAMILTON, Douglas James, B.Sc.(Eng.) South Africa, of Salisbury, S. Rhodesia.
HARRIES, Phillip John, of Rumney, Cardiff.
HOWARTH, George Brian, of Manchester.
JOLLEY, Dennis, of Bolton, Lancs.
JONES, Vyvyan Philip, B.Sc.(Eng.) London, of Wad Medani, Sudan.
KELSALL, George Stuart, A.M.I.Mun.E., of Lytham St. Annes, Lancs.
LEVELL, Donald John, B.Sc.(Eng.), London, A.M.I.Mun.E., of Portsmouth, Hants.
LEWIS, Frank Edgar, of London.
MACKENZIE, Ian Gilroy, of Colwall, Worcs.
MAGASINER, Alexander Sascha, B.Sc.(Eng.) Rand, of Johannesburg, South Africa.
MOHAMED, Shawky Hassan, of London.
MORLEY, Kenneth Outram, B.Sc.(Eng.) Cape Town, of Port Elizabeth, South Africa.
NAPIER, Malcolm Augustus, B.Sc.(Civil) Wales, of Aldermaston, Berks.
NASR, Mohmoud El-Sayed Ali, B.Sc.(Eng.) Egypt, of Giza, Egypt.
NEEDHAM, Frederick Harold, B.Sc.(Eng.) London, A.C.G.I., of Sanderstead, Surrey.
PARDOE, Frederick James, B.Sc.(Civil) Birmingham, of Birmingham.
PIGOTT, Edward Daniels, B.A., B.A.I.(Dublin), A.M.I.C.E., of Karachi, Pakistan.
REVESZ, Stephen, M.Sc.(Eng.) London, D.I.C., of Birmingham.
SMART, James Desmond, of London.
SUTHERLAND, George Athol, B.Sc.(Eng.) St. Andrews, A.M.I.Mun.E., of Crawley, Sussex.
TUCKER, Colin, of Sandbach, Cheshire.
YAP EE SIEW, B.Sc.(Eng.) Yale, of Singapore.

MEMBERS

FOSTER, Edward, of Gatley, Cheshire.
MOTION, John, of Bearsden. Dumbartonshire.

TRANSFERS

Students to Graduates

DRAGE, John Frederick, of Wembley, Middlesex.
EARP, Clifford David, of King's Newton, Derbyshire.
EVERETT, Frank Joseph, of Romford, Essex.
MARTIN, John Bernard, of London.
MORLEY, Charles Edward, of Liverpool.
PHILLIPS, William, of Newcastle upon Tyne.
SEEDHOUSE, Peter Duffell, of Birmingham.
WEBSTER, Donald Kenneth, of Bromley, Kent.
WILSON, George Thomas Eunson, B.Sc.(Civil) Edinburgh, of Edinburgh.

Associate-Members to Members

BAKHOUM, Michel, B.Sc.(Hons.), M.Sc., Ph.D., of Giza, Egypt.
BOTTOMLEY, Major John Hubert, of Hongkong.
COLLINS, Arthur Richard, M.B.E., D.Sc., A.M.I.C.E., of London.
FARQUHARSON, Douglas Gordon, B.Sc., A.C.G.I., A.M.I.C.E., of London.
HUGHES, John Ernest, of Harrow, Middlesex.
MCMEEKIN, Ronald Douglas, A.M.I.C.E., of Pinner, Middlesex.
MOHARRAM, Ahmed, M.Sc., Ph.D., of Giza, Egypt.

Members to Retired Members

ORRELL, Arthur Charles Victor, A.M.I.C.E., of Purley, Surrey.
SMITH, Herbert Brigg, of Baildon, Yorks.

RE-ADMISSION

Associate-Member

MCEWAN, Lt.-Col. James Baird, R.E., of St. Annes-on-Sea, Lancs.

OBITUARY

The Council regret to announce the deaths of VLADIMIR BENT, GEORGE CLAUDE CUTHBERT, MORRISON HENDRY, WILFRED ERNEST HUNT, ALEXANDER MCEWAN, WILLIAM DICK MCLAREN, HENRY AMBLER WHITAKER (Members); JOHN MILLAR, STANLEY VOILE NICHOLSON (Retired Members); HENRY GORDON HOPE, WALTER WILLIAM ROBERT ROCHE (Associate-Members).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of REUBEN LEONARD PERRY (Associate-Member) and AUBREY CROOKES (Graduate).

EXAMINATIONS

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on July 14th and 15th, 1953 (Graduateship), and July 16th and 17th, 1953 (Associate-Membership).

FORTHCOMING MEETINGS

Friday, May 1st, 1953

Annual Dinner, at the Dorchester Hotel, Park Lane, London, W.1, 7 o'clock for 7.30 p.m. The Principal Guest will be the Rt. Hon. Lord Sempill, A.F.C. Speeches will conclude at approximately 9.30 p.m., and will be followed by dancing until 2.0 a.m.

Thursday, May 14th, 1953

An Ordinary Meeting of the Institution will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 14th, 1953, at 6 p.m., when Mr. G. M. Frost (Member) will give a paper on "The Structural Aspect of Power Station Design in South Africa."

Members wishing to bring guests to this meeting are requested to apply to the Secretary for tickets of admission.

Wednesday, May 20th—Friday, May 22nd, 1953

Summer Meeting in Scotland. The Headquarters will be at the Royal Technical College, Glasgow, and members will have an opportunity of enjoying the hospitality of the Scottish Branch for certain items in the programme, particulars of which were circulated with the February issue of the Journal.

Thursday, May 21st, 1953

The Annual General Meeting of the Institution of Structural Engineers will be held at the Royal Technical College, George Street, Glasgow, on Thursday, May 21st, 1953, at 5.30 p.m.

The Annual General Meeting of the Voting Contributors to the Institution of Structural Engineers' Benevolent Fund will be held at the Royal Technical College, Glasgow, on Thursday, May 21st, 1953, at 5.45 p.m.

An Ordinary Meeting of the Institution will be held at the Royal Technical College, Glasgow, on Thursday, May 21st, 1953, at 6 p.m., when a paper entitled "Engineering Aspects of the New Atomic Research Extension, University of Glasgow," will be given by Mr. D. M. Hamilton, B.Sc., M.Eng., M.I.C.E., and Mr. J. Hardie Glover, A.R.I.B.A. Introduced by Professor W. T. Marshall, B.Sc.(Eng.), Ph.D., M.I.Struct.E., A.M.I.C.E.

Thursday, May 28th, 1953

Ordinary General Meeting for the election of members, 11, Upper Belgrave Street, S.W.1. 5 p.m.

Thursday, June 25th, 1953

Ordinary General Meeting for the election of members, 11, Upper Belgrave Street, S.W.1. 5 p.m.

REPRESENTATION

The Council have made the following nominations of members to represent the Institution :

ARCHITECTS' REGISTRATION COUNCIL OF THE UNITED KINGDOM AND ADMISSION COMMITTEE

Mr. F. R. Bullen (Hon. Curator).

REGIONAL ADVISORY COUNCIL FOR FURTHER EDUCATION IN THE SOUTH-WEST—NORTHERN AREA BUILDING ADVISORY COMMITTEE

Mr. C. J. D. Boxall (Member).

RESEARCH AWARDS

The Council have awarded Research Diplomas for 1951-52 to the following authors of papers published in THE STRUCTURAL ENGINEER :—

Dr. W. B. Dobie, for a paper on "The Torsional Strength of Structural Members."

Dr. S. Mackey (Associate-Member) and Dr. D. M Brotton (Graduate) for a paper on "An Investi gation into the Behaviour of a Riveted Plat Girder under Load."

REGULATIONS GOVERNING ADMISSION TO MEMBERSHIP

The Council have made the following amendment t "The Regulations Governing Admission to Membershi of the Institution" :—

Paragraph v of Appendix II, which read :

"The Higher National Certificate or Diploma in eithe Engineering or Building, if the subjects covered by th documents include Structural Engineering or Theory o Structures and either Engineering or Building Science,' is now to read :

"A Higher National Certificate or Diploma in eithe Engineering or Building if the subjects covered by th documents include

(1) Either Structural Engineering or Theory o Structures

and

(2) Either Engineering Science or Building Science. Both of the selected subjects must have been studied fo at least two years and the examination papers in these subjects must have been assessed in the final year i which the subject was taken."

DRURY MEDAL AWARD

The fourth competition for the above award will tak place in 1953. The subject is the design of the structur of a new factory building. The material of constructio is entirely at the choice of the competitor. The compe tition has been designed to encourage ingenuity structural arrangement. Economy in the use of steel an important feature of this year's competition.

Graduates and Students of the Institution who wis to compete are invited to apply for full details to th Secretary ; envelopes to be marked in the top left-han corner, "Drury Medal Award,"

The closing date for the competition is October 1s 1953.

The general conditions of the competition are follows :—

1. The competition shall be for Graduates an Students of the Institution of not more than 25 yea of age.

2. The subject of the competition shall be a design a structural character, that is to say primarily structur design, not planning.

3. The subject of design and conditions shall b prepared and issued biennially by a group of five mem bers appointed by the Council.

4. The Literature Committee shall appoint a Jury not less than five to examine the works submitted an to interview candidates, if found necessary.

5. In order to show that the work submitted is sole the work of the competitor, the documents submitte shall be countersigned by a corporate member of th Institution or, failing this, shall be accompanied by declaration on a prescribed form signed by the candida in the presence of a Justice of the Peace or a Commi sioner for Oaths.

EXAMINATIONS

PREPARATION FOR THE EXAMINATIONS OF THE INSTITUTION BY ATTENDANCE AT TECHNICAL COLLEG

A candidate for Graduateship or Associate-Membe ship may be able to attend a technical college ; the

notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees or Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges, courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in the first category provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

The Colleges which have informed the Institution that courses in Structural Engineering are available are :—

Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Salford Royal Technical College.
South-West Essex Technical College, Walthamstow, E.17.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in the second category provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete. The colleges which have informed the Institution that such courses are available are :—

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Plymouth and Devonport Technical College.
Preston, Harris Institute.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.

Students attending colleges in the first category are advised to take the organised courses in Structural Engineering. Students of Graduate Membership standard will usually be allowed to select subjects from courses provided by colleges in the second category.

LONDON GRADUATES' AND STUDENTS' SECTION

The following meeting has been arranged :—

Saturday, May 16th, 1953

Visit to Messrs. E. Sherry, Ltd., Barking Wharf, River Road, Barking, to see modern methods of timber seasoning and timber preparation and storage. Meet outside the Works at 10.15 a.m.

Hon. Secretary: J. F. S. Pryke, B.A.Hons., "Bushcroft," Slipe Lane, Wormley, Herts.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES BRANCH—GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES BRANCH

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., "Lisleen," 13 Finaghy Road North, Belfast.

SCOTTISH BRANCH

Hon. Secretary : D. G. Drummond, B.Sc., M.I.Struct.E. A.M.I.C.E., 11, Woodside Terrace, Glasgow, C.3.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meeting has been arranged :—

Wednesday, May 13th, 1953

Annual General Meeting at the Mackworth Hotel, Swansea, at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

The Fifth Meeting of the Session was held at the University of Bristol on February 6th, when Mr. F. G. Clarke (Associate-Member) presented a paper on "Some Local Contracts and Welded Steelwork for a Bus Garage."

The paper was well illustrated and the local nature of most of the jobs referred to were particularly interesting to members and their friends. Professor A. G. Pugsley, O.B.E., F.R.S., M.I.C.E., F.R.Ae.S. (Vice-President) proposed a vote of thanks to the author for his paper and for the very able manner in which he had answered the questions put to him.

An extra meeting was held at the University of Bristol on February 27th, when Mr. A. Mason, A.M.I.C.E. (Associate-Member) presented a paper on "Design and Construction of Reinforced Concrete Bridges with particular reference to Southern Rhodesia." The author illustrated the various types of bridges which have been constructed in Southern Rhodesia in recent years and it was interesting to note the improvements in methods of design and construction with each succeeding bridge. A vote of thanks to the author was proposed by Mr. C. E. Saunders (Member).

The Sixth Meeting of the Session took the form of Combined Meeting with the Institution of Civil Enginee and was held at the University of Bristol on March 5t when a paper on "Plymouth 'B' Power Station" w presented by Mr. L. Richardson, A.M.I.C.E. (Associat Member). A vote of thanks was proposed by Mr. C. Williams (Member).

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 2 Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 1 The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E A.M.I.Struct.E., P.O. Box 3306, Johannesburg. Duri week-days Mr. Tait can be contacted in the City E gineer's Department, Town Hall, Johannesbur 'Phone : 34-1111 Ext. 257.

Natal Section, Hon. Secretary : E. G. Bennet A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E P.O. Box 1692, Cape Town.

Book Reviews

Civil Engineering Reference Book, Edited by E. H. Probst, D.Eng., M.I.C.E. and J. Comrie, B.Sc., A.M.I.C.E (London : Butterworths Scientific Publications, 1951) 1,700 pp. 10 in. × 6½ in. £7 7s.

The aim of this book, as stated in the introduction, is to give to the civil engineer, in whatever capacity he may be acting, a concise presentation of the fundamentals of the theory and practice of all branches, and this purpose it fulfils admirably.

Divided into 48 chapters, each written by an expert in the particular field, every branch of civil engineering is covered. Some of the sections of particular interest to structural engineers are on mechanics, strength of materials, thin slabs and shells, theory of structures, soil mechanics, cements, site investigation, foundations and earthworks, steel and reinforced concrete bridges and buildings, welded structures, principles of plain and reinforced concrete, precast concrete construction, light alloy construction, timber construction.

There are chapters on subjects common to several branches of civil engineering, such as specifications and quantities, surveying, building materials and works construction, and short chapters on such special subjects as explosives, fire protection, legal notes and patents. A few chapters have been added dealing with developments abroad.

Fundamental theory is given in the chapters on general science, and many worked-out examples are included throughout the volume. Each section concludes with a useful bibliography to act as a guide to further study.

The book is well illustrated, and the general appearance, arrangement, paper and print are of high standard.

Telai Elastici, by Carlo Luigi Brunoli. (Milan : Ulrico Hoepli, 1951.) 223 pp. 10 in. × 7¾ in. Price 2,000 lira.

This book gives a method for the calculation of elastic building frames. It is divided into four parts, the first of which deals generally with vertical elastic frames using a method based on nodal points. The second part is confined entirely to Vierendeel girders and the thi deals with the torsion of horizontal frames. Part consists of worked-out numerical examples.

Dimensional Analysis and Theory of Models, Henry L. Langhaar. (New York : Wiley, London Chapman & Hall, 1951.) 166 pp. 9 in. × 6 in. 32s.

In this book Professor Langhaar has set out to co dense into a single volume of 166 pages the applicatio of dimensional analysis and model theory to the fo major branches of engineering science.

The first four chapters are devoted to the fundamen concepts of dimensional analysis and its applications, t various systems adopted for the calculation of dime sional products, and proofs of the underlying algebr theory. The development of the Buckingham theore for the reduction of a dimensionally homogenc equation among several variables is presented in chap four and should be studied by all those engaged experimental engineering research.

The author then deals with the laws of similarity a their application to the testing of model structur This is followed by four chapters devoted to the app cations of dimensional analysis to stress and strain, flu mechanics, theory of heat and electromagnetic theo Finally there is a chapter devoted to the derivation model laws from the governing differential equations.

The appeal of the book to structural engineers w largely be confined to those engaged on research work a on model structure analysis and in this restricted field t book suffers slightly from the wide field of applicatic covered.

In fairness to the author however, it must be stat that the book is primarily intended as a students te book. As such it fills a definite gap in engineeri literature, and in the development of engineering deg and diploma courses the book may be profitably stud by both teachers and students alike. The practisi engineer who may have occasion to apply dimensional model analysis to practical design will find the essenti of the subject presented in a clear and logical fashi which tends to easy reading. S.

Some Recent Foundation Research and its Application to Design*

By G. G. Meyerhof, Ph.D., M.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E., A.M.ASCE.

Summary

The bearing capacity of shallow and deep foundations under a central load is briefly discussed, and the effects of eccentricity and inclination of the load are indicated. The total and differential settlements of foundations are estimated in relation to the soil and foundation characteristics and the stiffness of the structure. The settlement stresses in foundation and superstructure are considered, and recent observations on brick walls and cladded building frames are analysed. The lateral movement and tilting of foundations are estimated and illustrated by the results of some loading tests on model portal frames on clay and sand.

Introduction

In the past, foundation design was essentially empirical and based largely on permissible bearing pressures or loads to avoid failure of the ground. The study of foundation behaviour in the field amplified by laboratory research has led to a more rational approach in which the results of a site exploration and soil tests form the basis of an analysis of the stability and probable movement of the proposed foundation and superstructure. The present paper is mainly concerned with recent research on foundation behaviour and indicates how the results may be applied to structural analysis.

The main purpose of the paper is to show how the soil, the foundation and the structure can be considered as one integral unit and to discuss in general terms the main aspects of this approach. Since foundations form the link between the superstructure and the soil, they have to fulfil two main interrelated functions : First, the stability of the foundation soil must be assured with an adequate margin of safety and, second, the movement of the foundation must be within limits that can be tolerated by the superstructure. It is convenient to discuss these aspects separately first and then to show how they may be interrelated.

1. Stability of Foundation

In order to ensure the stability of the structure, the loads transmitted to the foundation must be within the ultimate load bearing capacity of the soil. An estimate of this bearing capacity at which shearing failure of the soil occurs can be obtained from plastic theory[1, 2], using a number of simplifying assumptions. Thus the zones of shearing failure in the soil near the foundation are idealised as indicated in Fig. 1. In addition, the shearing strength of the soil is represented by

$s = c + p \tan \varnothing$ (Coulomb-Mohr's theory of rupture),

where c = cohesion,

p = normal pressure on shear plane,

and $\varnothing$ = angle of internal friction.

Further, the effect of the deformation of the soil on the bearing capacity is taken into account by an empirical reduction of the shearing strength[1, 2].

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The total bearing capacity of a foundation of base area A_b and side (or shaft) area A_s is then given by

$$Q = qA_b + s_a A_s \quad \ldots \quad (1)$$

where q = unit base resistance

and s_a = average unit skin friction.

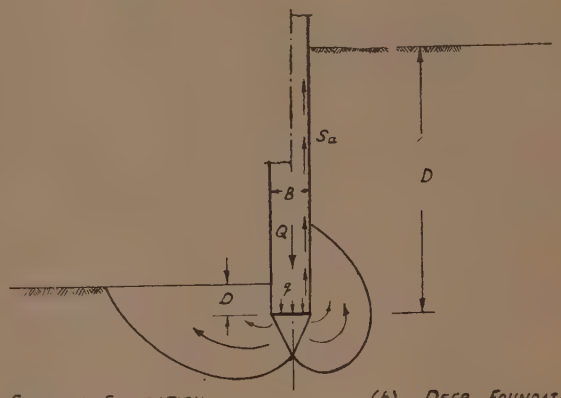


Fig. 1.—Theoretical failure zones at bearing capacity of foundations

It has been shown[2] that for a foundation of width B and depth D in uniform soil

$$q = cN_{cq} + \gamma \frac{B}{2} N_{\gamma q} \quad \ldots \quad (2)$$

where γ = density of soil

and N_{cq} and $N_{q\gamma}$ are bearing capacity factors, depending mainly on $\varnothing$, depth and shape of the foundation and the inclination of the load. The base resistance and skin friction can be estimated from the mechanical properties of the soil and ground water conditions and from the physical characteristics and method of installation of the foundation.

(a) *Shallow Foundations*

For footings and rafts when $D/B \leq 1$, the above expressions can be simplified because the skin friction is small and may be ignored. For clays (purely cohesive materials, $\varnothing = 0$) the bearing capacity is therefore given by the first term of eq. (2), while for sands and gravels (cohesionless materials, $c = 0$) the bearing capacity can be represented by the second term of eq. (2). The results of this analysis, which is supported by extensive research on model footings and limited field data[2, 3], are shown in the lower parts of Figs. 2a and b for a vertical load on a horizontal foundation. In the case of clays the original overburden pressure γD at base level can be added to the graphed values (Fig. 2a). Full submergence of sands and gravels reduces the effective density and thus the bearing capacity by about one-half (Fig. 2b) ; the bearing capacity is however not affected by a water table at a distance greater than

about 1.5B below the base so that the bearing capacity for intermediate positions of the water table can be interpolated accordingly.

In practice foundations are frequently subjected to a horizontal thrust H and bending moment M in addition to a vertical load V. This condition reduces the bearing capacity and can best be considered in two stages :— (i) the effect of inclination, and (ii) the effect of eccentricity of the load on the bearing capacity. The following results have been obtained from a theoretical analysis, which is supported by loading tests on model footings[4].

For a load that is inclined at an angle $\alpha = tan^{-1}(H/V)$ to the vertical, the factors N_{cq} and $N_{\gamma q}$ in eq. (2) are reduced, and for any position of the load on horizontal foundations the reduction factors given in Table 1 have been obtained.

The bearing capacity of an eccentric inclined load is readily estimated by combining the methods of analysis summarised above, i.e., the reduction factors for inclination are used in combination with the effective contact width or area for eccentricity. In the case of pad footings the whole structural H, M, and V are transmitted to the horizontal bearing area since the soil resistance on the vertical sides is negligible. In the case of deep block footings (D/B approaching 1) with large eccentricities (e/B approaching 0.5) however the resistance on the sides enables somewhat larger forces to be applied to the top of the footings, especially in cohesionless soils.

So far as building frames on single column footings are concerned, it follows that in the absence of structural failure the maximum H, M, and V which can be imposed

TABLE 1.—Reduction Factors for Vertical Bearing Capacity of Shallow Horizontal Foundation under Inclined Load

Soil Type	Foundation Depth / Width D/B	Inclination of load with vertical α						
		0°	10°	20°	30°	45°	60°	90°
Clays	0 to 1	1.0	0.8	0.6	0.4	0.25	0.15	0
Sands and Gravels	0	1.0	0.5	0.2	0 for $\alpha = \phi$		—	—
	1	1.0	0.6	0.4	0.25	0.15	0.05	0

The vertical component of the bearing capacity at any given inclination of the load is then found by multiplying the bearing capacity for a vertical load (e.g., Figs. 2a and b for a central load) by the appropriate reduction factor. The horizontal component of the bearing capacity is equal to tan α times the vertical component ; to this result can be added the passive resistance on the vertical face of the toe.

If the load on a foundation acts at an eccentricity $e = M/V$, the maximum toe pressure has hitherto been taken to govern the bearing capacity. This approach is generally safe for small eccentricities but not for large ones[4]. A more rational method is obtained by assuming that the contact pressure distribution under an eccentric load is identical with that of a centrally loaded foundation of reduced width B'. Thus for a strip foundation with single eccentricity e of the load acting in any direction

$$B' = B\ (1 - 2e/B) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

so that for clays the bearing capacity

$$q = q_c\ (1 - 2e/B) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4a)$$

while for cohesionless soils

$$q = q_c\ (1 - 2e/B)^2 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4b)$$

where q_c = bearing capacity of a central load (e.g. Figs. 2a and b for a vertical load) at given D/B'.

With double eccentricities e_x and e_y of the load, the minimum effective contact area A'_b (with straight boundary across the base) is found by assuming that the centroid of the area coincides with that of the load. Then the total bearing capacity

$$Q = qA'_b \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

where q is given by eq. (2) but now depends on B' instead of B.

on a column at footing level are interdependent and are governed by the bearing capacity of the soil for the particular ratios of H/V and M/V. Thus for a given H/V and a single eccentricity of the load the maximum bending moment at the column base is

$$M = Q_c e\ (1 - 2e/B) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6$$

where Q_c = total bearing capacity of central load (vertical or inclined). In conjunction with the previous relationships it can be shown that for clays the absolute maximum $M = Q_c B/8$ independent of depth, while for sands and gravels the maximum M varies from $Q_c B/13.5$ for $D/B = 0$ to about $Q_c B/10$ for $D/B = 1$. Similarly, for a given M/V the maximum shearing force H at the column base depends on α ; and it can be shown that for clays the absolute maximum H varies from about $0.2Q_v$ for $D/B = 0$ to $0.4Q_v$ for $D/B = 1$ while for cohesionless soils the maximum H varies from about $0.1Q_v$ for $D/B = 0$ to $0.2Q_v$ for $D/B = 1$, where Q_v = bearing capacity of vertical load (central or eccentric).

The whole analysis in this section applies strictly only to single footings but is also sufficiently accurate for groups of shallow footings and for rafts. Where the soil is variable with depth, the average shearing strength within a depth of about $1\frac{1}{2}B$ below the base, controls the bearing capacity at foundation level provided the strength at a depth of $1\frac{1}{2}B$ is not less than about one third of the average upper shearing strength ; if not, then the bearing capacity is governed by the weaker lower layer and can be estimated by spreading the load to that layer at an angle of 30° to the vertical.

(b) *Deep Foundations*

For piers and piles the skin friction (adhesion) may form the major contribution to the bearing capacity depending on the soil type and the method of installing the foundation. In clays, the adhesion varies between wide limits and, except in the case of piers, governs the bearing capacity. Thus field loading tests on bored

-situ piles installed in an unlined auger borehole in iff London clay showed that water from the concrete ftens the clay and reduces the adhesion to 300-)0 lb./ft.^{2} ; on the other hand tests on driven piles, even very sensitive clays, indicated that after a month or so ie adhesion may be taken as equal to the undisturbed iearing strength of the soil within a limit of about)00 lb./ft.^{2}(2). These lower and upper limits of the :aring capacity are indicated in Fig. 2a.

In sands and gravels, however, the skin friction can :ually be ignored and the corresponding bearing ipacity is shown in Fig. 2b. Where piers or piles pass irough clay and rest in underlying cohesionless material, ie skin friction of the clay (Fig. 2a) can be added to ie base or point resistance of the cohesionless soil

not exceed one-third of the ultimate bearing capacity under normal loading conditions nor one-half of the ultimate under the most unfavourable conditions, when substantial local overstressing of the soil near the foundation has to be avoided. These factors of safety are of the same order as those normally used in structural design ; since the most unfavourable loads act only for a short time and do not affect the settlement, settlement estimates should always be based on the normal loads in order to decide whether a factor of safety of 3 against shear failure is adequate to keep the movements within tolerable limits.

While the bearing capacity of most types of foundations can be assessed from simple soil tests with an accuracy sufficient for most practical purposes, the

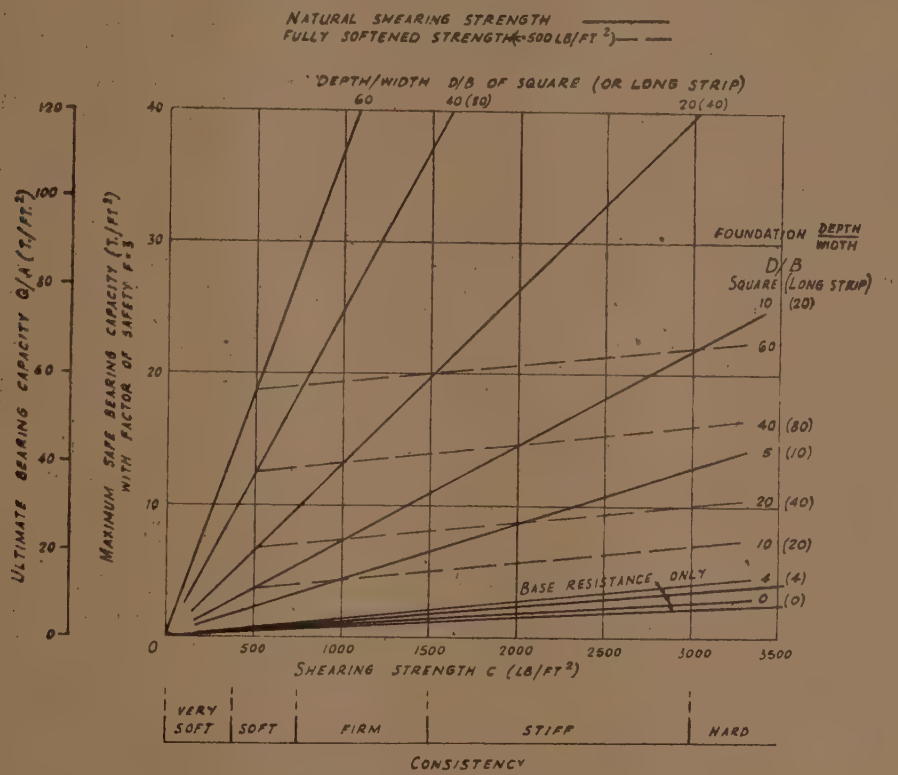


Fig. 2 (a).—Bearing capacity of foundations in clays

rresponding to the appropriate embedment ratio ig. 2b). On account of the small point resistance in ays, piles should generally not be driven through sand gravel layers, which also assist in reducing settlement spreading the load.

Where the pier or pile spacing is less than about four mes the base or point width, the bearing capacity of a oup is usually given by the base resistance of piers or les and enclosed soil plus the skin friction of the rimeter of the group ; for stratified soil the method of timating the bearing capacity is similar to that tlined above[5]. The bearing capacity of piers or pile oups under eccentric and inclined loads can be esti-ated as indicated for shallow foundations with allow-nce for any lateral soil resistance[1].

2. Settlement of Foundation

If the foundation is located below the depth at which e soil is subject to volume changes due to climatic artificial influences, the movements due to structural ads form the criterion to the permissible bearing essures or loads. To ensure a reasonable margin of fety against shear failure of the soil, it has been ggested[6] that the permissible foundation loads should

probable settlement, and even more, its distribution over the foundation area depends on so many factors that it can only be estimated within fairly wide limits except in special cases. In general, the average settlement of a structure carrying a given load decreases as the size and depth of the foundation increases. Since stronger soils are usually less compressible than weaker ones, it follows that the settlement of a foundation is likely to decrease as the bearing capacity of the soil increases, other things remaining unchanged. For soils which are uniform with depth the above relationships can be expressed quantitatively within certain limits.

(a) *Total Settlement*

The settlement of a structure due to foundation loads can be divided into (i) immediate settlement due to deformation of the soil without volume change, and (ii) consolidation settlement due to closer packing of the grains. The immediate settlement occurs mainly on application of the load and consists of an elastic (recoverable) portion, which can be estimated from elastic theory[7], and a plastic (irrecoverable) portion, which can be estimated from plastic theory[8]. In practice it is how-

ever frequently sufficiently accurate to estimate the whole immediate settlement S_1 from elastic theory, using a mean secant modulus of deformation E_s at one-half of the compressive strength of the soil[3]. Then it can be shown that for a foundation of width B and bearing pressure p

$$S_1 = \frac{3\,C\,p\,B}{4\,E_s} \quad \ldots\ldots\ldots\ldots \quad (7)$$

where C = influence value depending mainly on shape and depth of foundation and thickness of compressible stratum.

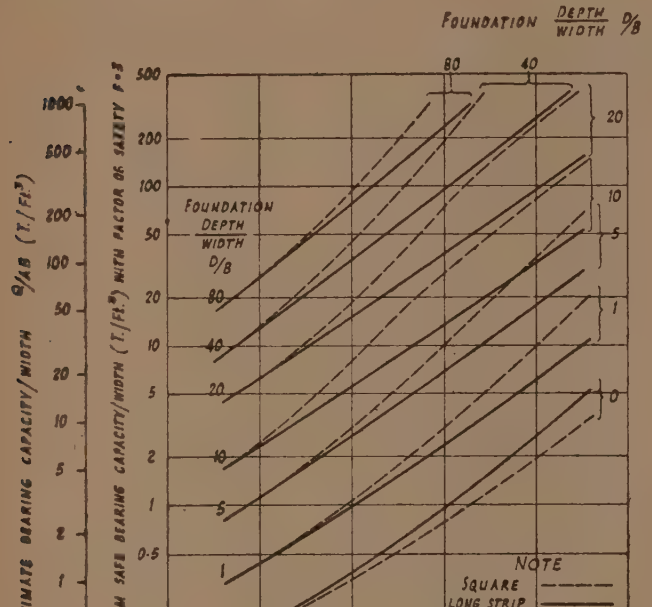


Fig. 2 (b).—Bearing capacity of foundations in sands and gravels

Analysis of field observations on spread foundations in clays indicates[3] that E_s varies from about 50 to 200 times the cohesion and approximately

$$S_1 = (2 \text{ to } 8\%)\ B/F \quad \ldots\ldots\ldots \quad (8)$$

where F = factor of safety against shear failure ($F \geq 3$).

The magnitude of consolidation settlement S_t can be estimated from consolidation theory[1] and may be represented by

$$S_t = \frac{C\,p\,U\,B}{K_v} \quad \ldots\ldots\ldots\ldots \quad (9)$$

where K_v = mean modulus of compressibility of soil and $0 < U \leq 1$ is the degree of consolidation at any given time.

The maximum time to be considered depends on the economic life of the structure, and in preliminary estimates U can be taken as unity.

Combining eqs. (7) and (9) the total settlement

$$S = S_1 + S_t = \frac{C\,p\,B}{E'_s} \quad \ldots\ldots\ldots\ldots \quad (1$$

where E'_s = effective modulus of deformation

$$= \frac{E_s\,K_v}{0.75\,K_v + UE_s} \quad \ldots\ldots\ldots\ldots \quad (1$$

Preliminary examination of published data indicate that K_v varies from about 25 to 80 times the cohesi for normally-consolidated (i.e., recent and general soft) clays to from 70 to 200 times the cohesion for ove consolidated (older and usually firm to hard) clays. the latter case the order of the observed total settleme $(U \geq 1)$

$$S = (3 \text{ to } 7\%)\ B/F \ (F \geq 3). \quad \ldots\ldots\ldots \quad (1$$

For normally-consolidated clays the estimated settleme would be up to three times larger. Further, the limit field observations indicate that on the same clay t total settlement is up to two to three times the immedia movement.

On account of the large permeability of cohesionle soils, foundations on sands and gravels settle main on application of the load, and for practical purpos only the total movement is required. The results laboratory and field loading tests and settleme observations on buildings with shallow foundations sand indicate[6] that approximately

$$S = (20 \text{ to } 40\%)\ \frac{B^3}{(B+1)^2 F} \text{ (ft.)}\ (F \geq 3). \quad \ldots \quad (1$$

where B is measured in feet and the water table is more than B below base level ; if the water table is base level, the settlement is doubled.

The above relationships (eqs. 12 and 13 with F = enable a rough preliminary estimate to be made of t probable settlement of a structure on shallow found tions. After the effect of this movement on the structu has been ascertained along the lines indicated below more accurate assessment can then be made from detailed investigation of the particular site.

(b) *Differential Settlement*

The total settlement is mainly important in relati to access and services of a structure and is sensil unaffected by structural stiffness so that it rar influences structural design. The differential settleme however depends not only on the factors governing t total movement but also on the layout and rigidity the structure and the variation of the compressibil of the soil in plan ; it is therefore more difficult estimate and is more important because it affects t behaviour of the structure under working conditions

Analysis of field observations[6] indicates that t (maximum) differential settlement of footings on n mally-consolidated clays at customary bearing pressu may be very large (several inches) but for over-cons dated clays rarely exceeds $\frac{3}{4}$ in. For rafts on clay t differential settlement is about one-half of the to movement, but the slopes of the bowl-shaped deform tion are so gentle that the differential movement betwe adjacent columns never exceeds a small fraction of differential settlement of the whole raft. Settlem records of uniformly loaded strip footings and equa loaded pad footings of similar size on sands show t the (maximum) differential movement is unlikely

xceed three-quarters of the maximum settlement[6]. For afts on sand the differential settlement between adjacent olumns may be equal to the differential settlement of he whole raft, which is about three-eighths of the total novement.

If the total settlement has been calculated, then on he basis of these field indications the differential ettlement of a structure on shallow foundations can be stimated approximately ; and it has been suggested[6] hat most ordinary structures can withstand a differntial movement of $\frac{3}{4}$ in. between adjacent columns, rom which permissible bearing pressures have been elected. The author considers however that the novements which can be tolerated by a structure can nly be decided in each particular case on the basis of a tructural analysis. In an earlier paper a method was roposed[9] in which the foundation and settlement haracteristics of the soil at a site could be interrelated vith the loading, layout and rigidity of building frames n single footings. This method, which treats the superstructure, its foundations and the underlying soil as one omplete statically-indeterminate system, will now be xtended to give at least a qualitative indication of the ehaviour of different types of structures on different ypes of soil.

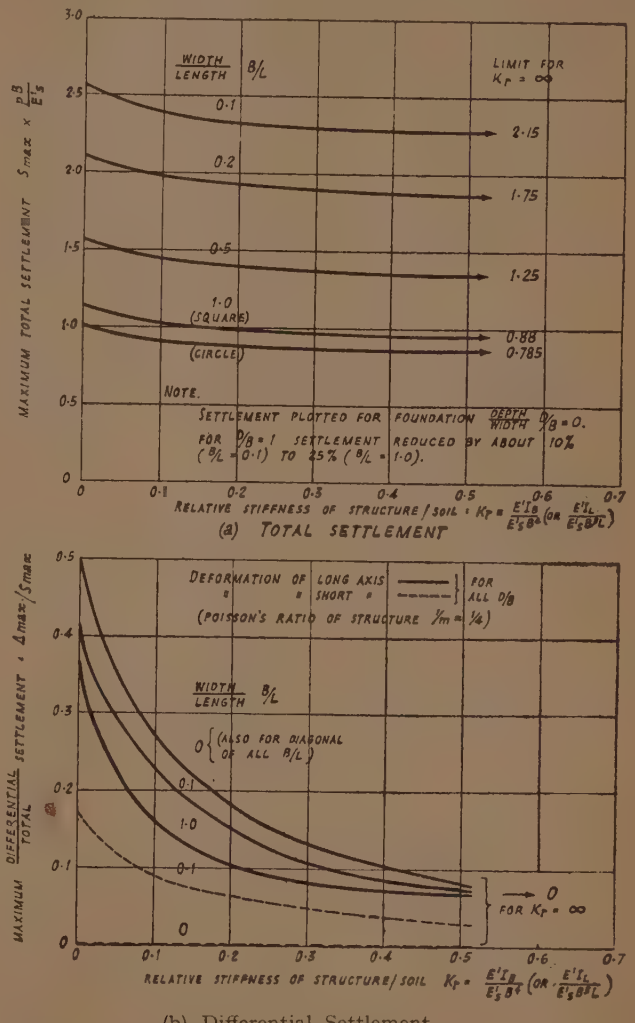
The present analysis is based on elastic theory for oth structure and soil, which would appear to be easonable for working stresses of one-third or less of the ltimate. The structure is assumed to be uniformly oaded with an effective flexural rigidity $E'I$ which llows for elastic and creep deformations. The soil is ssumed to be an isotropic half-space with a constant ffective modulus of deformation E'_s which includes lastic, plastic and consolidation characteristics (eq. 11). his analysis applies therefore mainly to foundations n cohesive soils ; in cohesionless materials E'_s increases vith depth so that the results are then only very approximate.

When a uniformly loaded foundation of width B ests on an elastic medium, it has been shown[10, 11] that he distribution of contact pressure on the base depends n the relative stiffness K_r of the foundation ($E'I$ per nit width or length) to that of the soil (expressed by E'_sB^3). Using the corresponding general equations for ircular and long strip foundations and the finite differnce approximation suggested[12] for other shapes, the uthor has computed the settlement and the bending noments and shearing forces (see below) along the rincipal axes of foundations of different shapes and elative stiffness K_r. Details of this analysis are outside he scope of this paper, but the general results obtained or the maximum total and differential settlements are ndicated in Figs. 3a and b. The total settlement ncreases with the length/width ratio L/B of the foundaion, but for any given L/B the (maximum) total settlement is not very much affected by the relative stiffness K_r (Fig. 3a). It can be seen from Fig. 3b that the maximum) differential settlement however decreases apidly as the relative stiffness K_r increases and, for a given K_r, increases somewhat with the ratio L/B. The settlement distribution along the principal axes of oundations was found to vary from an approximately ircular form for $K_r = 0$ to a parabolic form as K_r ncreased.

In practice the flexural rigidity of the whole foundation s frequently zero (i.e., where a building rests on a umber of individual footings) or is at least small ompared with that of the superstructure, especially here the latter consists of a rigid frame with cladding r of load-bearing walls. It is therefore necessary to btain at least an approximate estimate of the flexural rigidity $E'_B I_B$ of the superstructure, which has to be added to $E'_F I_F$ of the foundation to obtain the total $E'I$ on which the differential settlement depends. For this purpose two main types of superstructures, multistorey building frames and load bearing walls, will be considered.

Consider an open multi-storey building frame of length L with approximately equal bays of length l deflecting into the shape of a trough with a similar curvature at each floor level and a maximum differential settlement $\triangle$.

As the beams (including floor slabs in an encased steel or reinforced concrete frame) are generally stiff


MAXIMUM TOTAL SETTLEMENT $S_{max} \times \frac{p_B}{E'_s}$
WIDTH / LENGTH B/L
LIMIT FOR $K_r = \infty$
0·1
2·15
0·2
1·75
0·5
1·25
1·0 (SQUARE)
0·88
(CIRCLE)
0·785
NOTE.
SETTLEMENT PLOTTED FOR FOUNDATION DEPTH / WIDTH D/B = 0.
FOR D/B = 1 SETTLEMENT REDUCED BY ABOUT 10% (B/L = 0·1) TO 25% (B/L = 1·0).
RELATIVE STIFFNESS OF STRUCTURE / SOIL $K_r = \frac{E'I_B}{E'_s B^4}$ (OR $\frac{E'I_L}{E'_s B^2 L^2}$)
(a) TOTAL SETTLEMENT
MAXIMUM DIFFERENTIAL SETTLEMENT / TOTAL SETTLEMENT Δ_{max}/S_{max}
DEFORMATION OF LONG AXIS
" SHORT "
FOR ALL D/B
(POISSON'S RATIO OF STRUCTURE $\nu_m = \frac{1}{4}$)
WIDTH / LENGTH B/L
0 (ALSO FOR DIAGONAL OF ALL B/L)
0·1
1·0
0·1
0
0 FOR $K_r = \infty$
RELATIVE STIFFNESS OF STRUCTURE / SOIL $K_r = \frac{E'I_B}{E'_s B^4}$ (OR $\frac{E'I_L}{E'_s B^2 L^2}$)

(b) Differential Settlement

Fig. 3.—Effect of relative stiffness on total and differential settlements of uniformly loaded structures and foundations

compared with the columns, the points of contraflexure in the columns can now as a first approximation be taken at mid-storey height. Then it can be shown (Appendix 1a) that in any storey of height h the effect of the columns is to increase the flexural rigidity $E'I_b$ of the whole beam line to

$$E'I'_b = E'I_b \left[1 + \left(\frac{K_1 + K_u}{K_b + K_1 + K_u}\right) \frac{L^2}{l^2}\right] \quad . \quad (14)$$

where $K_b = I_b/l$ = average stiffness of beams,
$K_1 = I_1/h_1$ = average stiffness of lower columns
and $K_u = I_u/h_u$ = average stiffness of upper columns

for a rigid frame or the corresponding equivalent stiffnesses in a semi-rigid frame[9, 13].
Hence for the total superstructure

$E'I_B = \Sigma E'I'_b$ of each storey level obtained from eq. (14).

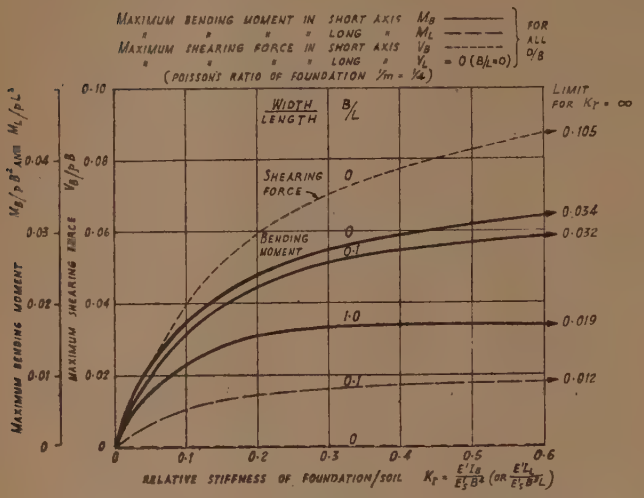


Fig. 4.—Effect of relative stiffness on maximum bending moment and shearing force in uniformly loaded foundations

As a check, this method has been applied to a 5-storey, 3-bay rigid frame, which had previously been analysed in detail[9]. Using eq. (14) it was found that the columns increased the beam stiffnesses about five times and the resulting differential settlement was within 5 per cent. of that obtained before.

External building frames are generally stiffer than indicated above on account of wall cladding. Where this consists of solid panels between the beams and columns and only shearing stresses can be transmitted from the framework to the panels, it can be shown (Appendix 1b) that in any storey the flexural rigidity (eq. 14) is increased further by

$$\frac{E'_p I_p L^2}{2\ h^2} \quad . \quad . \quad (15)$$

where $E'_p I_p$ = flexural rigidity of panel (in vertical plane). If the wall cladding is fully continuous so that the whole frame behaves like a solid deep beam, th flexural rigidity of the framework itself can frequent be ignored compared with that of the wall. The proble is then similar to that of load bearing walls, when i the absence of openings

$$E'I_B = \frac{E'b\ H^3}{12} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (1$$

where b = average thickness of wall
and H = height of wall or structure.

Openings in the cladding or walls reduce the flexur rigidity and require a more elaborate analysis.

3. Settlement Stresses in Foundation and Superstructure

After the differential settlement of the foundatio has been estimated, it is possible to obtain the stress in the foundation and superstructure from norm structural analysis. For a uniformly loaded foundatio the bending moments and shearing forces can howev be related directly with the relative stiffness K_r by th analysis mentioned above. The maximum bendi moments and shearing forces increase rapidly with K from zero for a perfectly flexible foundation to a max mum for a perfectly rigid foundation, and they increa with the ratio L/B (Fig. 4). The variation of th bending moment along the principal axes of foundatio follows that of the settlement, while the shearing for increases from zero at the edges of the foundation to maximum in the outer quarter sections and then d creases to zero at the centre of the area. This variatio is shown in Fig. 5 for a rigid strip foundation for which closed solution has been obtained (Appendix 2a).

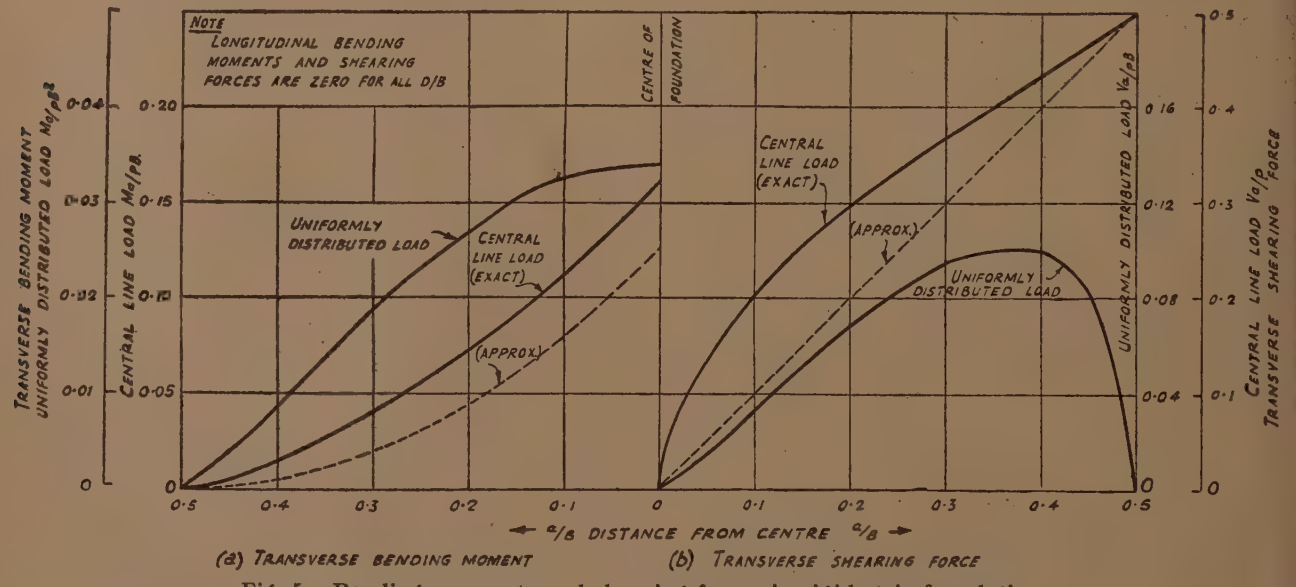

NOTE
LONGITUDINAL BENDING MOMENTS AND SHEARING FORCES ARE ZERO FOR ALL D/B
CENTRE OF FOUNDATION
TRANSVERSE BENDING MOMENT UNIFORMLY DISTRIBUTED LOAD M_B/pB^2
CENTRAL LINE LOAD M_B/pB
UNIFORMLY DISTRIBUTED LOAD
CENTRAL LINE LOAD (EXACT)
(APPROX)
UNIFORMLY DISTRIBUTED LOAD V_B/pB
CENTRAL LINE LOAD V_B/p TRANSVERSE SHEARING FORCE
$^a/_B$ DISTANCE FROM CENTRE $^a/_B$
(a) TRANSVERSE BENDING MOMENT
(b) TRANSVERSE SHEARING FORCE

Fig. 5.—Bending moments and shearing forces in rigid strip foundation

The above results, which are independent of found tion depth, can be applied directly to the design uniformly loaded rafts and solid load bearing wal when the stiffness and thus the bending moments an shearing forces can vary within wide limits. On th other hand single footings, unless very wide, may b taken as perfectly rigid ($K_r = \infty$). The correspondin bending moments and shearing forces in a strip footin with a central line load (Appendix 2b) are shown i Fig. 5 and differ little from those obtained by th

ustomary method based on a uniform distribution of he contact pressure.

Except in the case of solid walls just mentioned, the ending moments and shearing forces in superstructures re best ascertained from the estimated differential ettlements. The necessary equations were given reviously[9] for open (bare or encased) rigid and semi-igid building frames. Analysis of a few typical frames ndicated that considerable settlement stresses could e induced in all structural members and would be largest n beams at external joints and in all but centre columns, specially in the lower storeys. It was also shown that n order to prevent overstressing (as distinct from ollapse) the permissible bearing pressures or loads for a given layout of the foundation had to be reduced as the tructure became stiffer. Panels and other wall cladding tiffen exterior building frames appreciably, and the roportion of the load carried by the cladding and the ramework depends on their relative flexural rigidity, which in simple cases can be assessed from eqs. (14) and 15); the corresponding bending moments and shearing orces in the components follow then from the analysis Appendix 1). In practice this composite behaviour of he structure is much more complex, and in order to btain some idea of the order of the stresses induced by he deformation of structures, the results of recent esearch will be briefly analysed in relation to load earing brick walls and building frames.

Some vertical loading tests have recently been carried ut at the Building Research Station[14] on 8 ft. high olid brick walls without supporting beams on an 0 ft. 6 in. effective span. Cracking started at a central eflection of 0.004 in. ($\triangle/L$ = 1/20000) when the aximum tensile stress was about 50 lb./in.^{2}; failure id not even occur at three times the cracking load. In imilar tests on walls without and with central openings n shallow reinforced concrete supporting beams no racking was noticed at the previous maximum load when the deflection varied from 0.02 to 0.03 in. ($\triangle/L$ = 1/4000 to 1/6000) and the maximum stress was about 20 lb./in.^{2} in the solid walls. In loading tests[15] on 0 ft. high and 30 ft. long lightly reinforced brick walls with openings on concrete strip footings, which were acked up either at the centre or the ends of the walls, racking began at a central deflection of 0.05 to 0.08 in. ($\triangle/L$ = 1/4500 to 1/7000) when the average tensile tress (ignoring the effect of the openings) varied from 40 to 50 lb./in.^{2}; failure took place at about twice the racking load. It may be concluded from these pre-iminary experiments that brick walls on footings under hort-term loading are likely to crack if $\triangle/L$ exceeds 1/5000 (height/length ratio H/L from 1 to 3) and the ensile stress exceed 50 lb./in.^{2}. Under field conditions the $\triangle/L$ ratio for a similar stress is likely to be much greater due to creep of the brickwork. Thus from bservations on brick walls of buildings it was found[16] hat the maximum stresses at cracking were of the rder of 60 lb./in.^{2} and the corresponding angular distortion was 3.5 × 10^{-3} rad. with $\triangle/L$ of about 1/1000. In order to have some margin of safety, a maximum differential settlement $\triangle/L$ of 1/2000 and tensile stress of 30 lb./in.^{2} may thus be suggested for buildings with load bearing brick walls or continuous brick cladding.

Very little information exists about the distortion which encased steel and reinforced concrete frames can withstand without harmful cracking. Based on the common limitations of deflections of structural members to about 1/300 of the span l to avoid cracking of finishes, the limit δ/l = 1/300 may be suggested for the permissible differential settlement between adjacent columns of open frames, and a maximum tensile stress of, say, two-thirds of the tensile strength of the concrete. The stiffening and strengthening effect of wall panels in frames is shown by some (horizontal) racking tests carried out at the Building Research Station[17] on about 10 ft. high and 12 ft. long cased steel frames with solid brick, clinker and clay block infilling panels. Cracking of the panels was first noticed when the (horizontal) displacement varied from 0.3 to 0.4 in. (δ/l = 1/300 to 1/400) compared with 1 in. (δ/l = 1/100) at cracking of

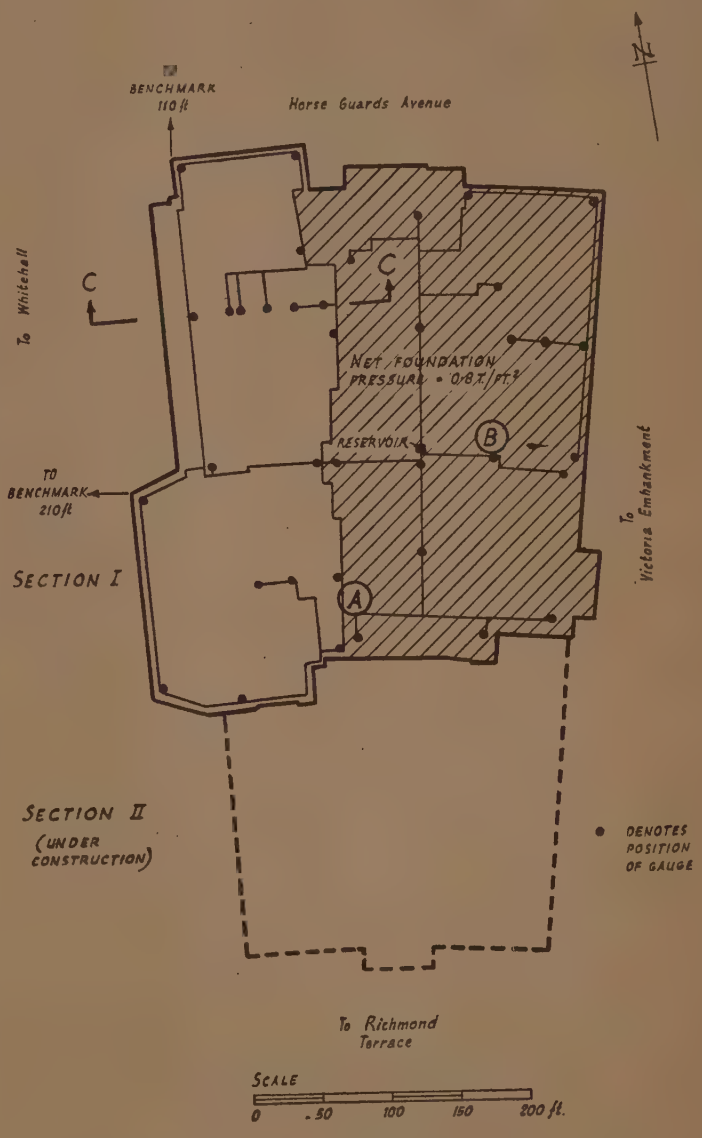


Fig. 6 (a).—Plan of Whitehall Gardens, London, showing layout of water levelling system

the encasement of a similar open frame; the corresponding average diagonal tensile stress in the panels appeared to vary from about 80 to 100 lb./in.^{2}. The maximum load was about 1½ times the cracking load and for the cladded frames it was 1½ to nearly 3 times that of the open frame. In other racking tests[18] on light frames with and without wall cladding for two-storey houses the framework took only a few per cent. of the total applied load with a corresponding reduction of the maximum horizontal deflection. Until further evidence is available, the differential settlement between adjacent columns of building frames with wall panels should therefore be limited to δ/l = 1/1000 and the maximum tensile stresses as suggested above for walls and concrete.

Thus, provided the permissible structural stresses are not exceeded, the limit

$$\triangle/L \text{ (or } \delta/l) = X$$

where X = constant depending on type of superstructure
and L = length of wall or l = column spacing, as the case may be.
It has also been shown that

$$\triangle/S \text{ (or } \delta/S) = Y$$

where Y = constant depending mainly on the relative stiffness K_r of structure to soil,
and approximately, the total settlement

$$S = ZB/F$$

where Z = constant depending mainly on soil conditions. Hence, substituting, the required factor of safety on

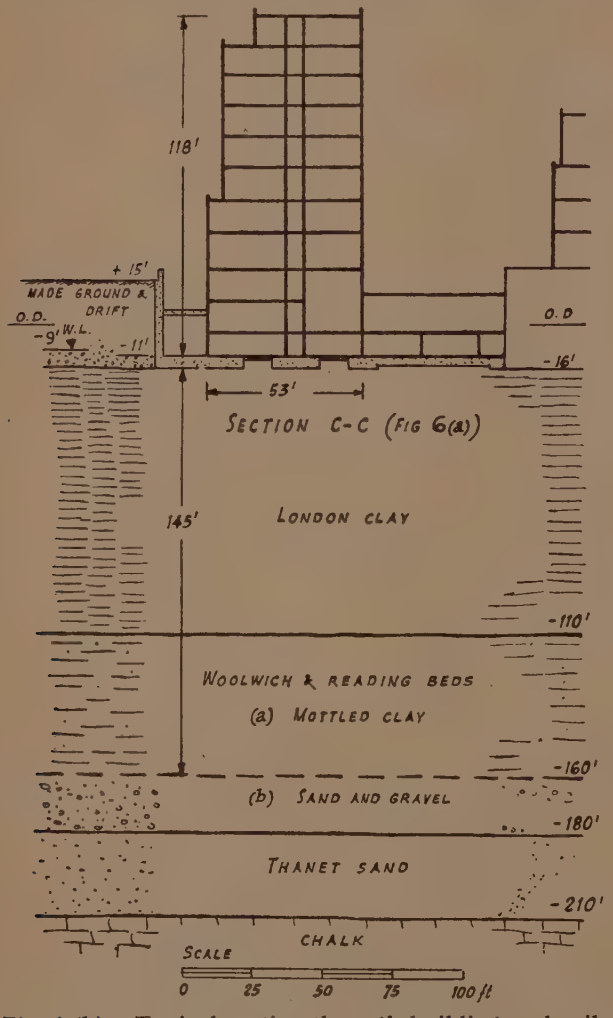


Fig. 6 (b).—Typical section through building and soil

the ultimate bearing capacity of the soil to avoid excessive deformation of the structure is

$$F = \frac{YZB}{XL} \quad \left(\text{or } \frac{YZB}{Xl} \right) \quad . \quad . \quad . \quad . \quad . \quad (17)$$

which depends on the characteristics of the soil, the foundation and the superstructure. This factor depends thus not so much on the breadth as on the shape in plan of the structure.

To provide information about the behaviour of framed buildings undergoing settlement, some observations have been made on a new Government Office Block, Whitehall Gardens, London; the results obtained, so far, illustrate many of the points made in the last two sections and will therefore be briefly described. This building, of which construction details have already been published[19], is a fairly stiff cladded steel-framed structure on a raft foundation of variable thickness resting mainly on London clay (Figs. 6a and b). Before construction a settlement estimate was made on the assumptions that the whole structure was perfectly flexible and rests on a 145 ft. thick compressible stratum whose settlement characteristics were the same as those determined at Waterloo Bridge[20]. On account of the great foundation depth, the foundation pressure was practically balanced by the overburden pressure except in the north-eastern portion (Fig. 6a). For the balanced areas where a fairly flexible raft was provided, it was estimated that during excavation the clay surface would rise by up to 2 in., which would largely be recovered during construction; subsequent movement due to the adjacent net loaded area was expected to be small. In the north-eastern portion, however, the superstructure was built on an existing stiff raft foundation and produced an average net bearing pressure of 0.8 $t/ft.^2$ with consequent long-term settlement; estimates for the edge (minimum at point A, Fig. 6b) and centre (maximum at point B, Fig. 6b) of that area are given in Fig. 6c.

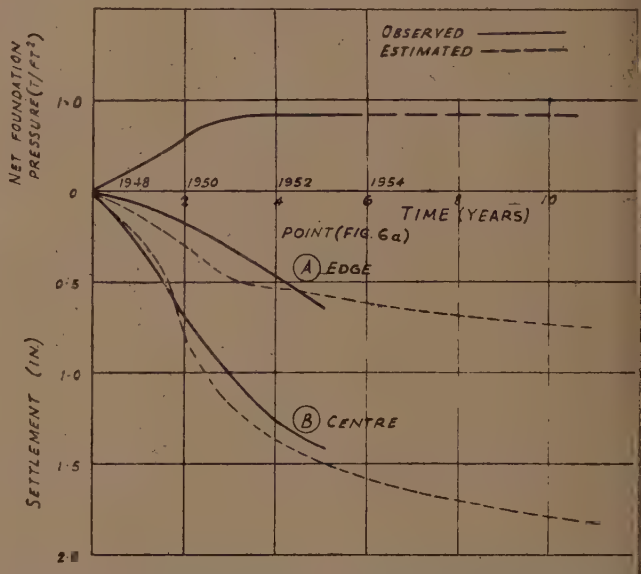


Fig. 6 (c).—Minimum and maximum settlements of points on north-western portion

During and after construction of the building (Fig. Section) settlement observations were made, first by ordinary levelling and later by means of a permanent water levelling system, installed on the sub-basement floor, with reference to outside benchmarks on existing buildings for the absolute movement (Fig. 6a). The installation consisted of ½ in. dia. copper tubes along outside walls and principal corridors with 5 ft. high stand-pipes where the water level could be measured on graduated glass tubes to determine differential settlements with an accuracy of 1/50 in. The recorded total settlement to date and the relative settlement since substantial completion of the north-eastern and north-western portions are given in Figs. 7a and b. The former portion on a relatively stiff raft has tilted slightly without appreciable distortion on account of the great total stiffness. The maximum settlement of about 1½ in. occurred near the northern end where construction began and the minimum of about ⅝ in. at the southern end; these movements are of the same order as those predicted (Fig. 6c). The north-western portion on a fair

exible foundation has assumed the bowl-shaped settlement distribution expected on theoretical grounds with maximum of about 2 in. as estimated.

At a section of the latter portion (*C—C*, Figs. 6a and b), where the maximum differential settlement between adjacent columns since substantial completion is of the order of 3/16 in. on a span of 25 ft. (Fig. 7b), an estimate has been made of the corresponding bending stresses induced in the structural members of the two bottom storeys on the assumption of rigidly connected bare sections with stiffnesses ($K = I/l$ or I/h) given in Fig. 7c. These estimated settlement stresses at the ends of members, where they are largest, do not exceed about $1\frac{1}{4}$ t/in.² (Fig. 7c), which is small compared with customary design stresses (8 t/in.²) for the dead and live loads on a bare frame. It is of interest to compare these estimates with the stresses measured by the Building Research Station using vibrating wire gauges fixed to the steelwork. The stresses observed since substantial completion (Fig. 7c) are generally of the same sign as those predicted from differential settlement, but they are on the whole much smaller (the maximum observed stress is, however, about $\frac{3}{4}$ t/in.² compared with an estimate of 1 t/in.² at the same section). This difference may be explained by the effect of cladding of the frame, which is particularly stiff in the first bay where 9 in. thick brick wall panels are provided. This composite behaviour is also borne out by the observations that on substantial completion of the building the maximum stresses were of the order of 3 t/in.², while in a recent loading test on a beam (fourth span, Fig. 7c) the measured maximum stresses and deflections were only about one-third of those estimated for a bare frame. It can therefore be concluded that although the present settlement stresses are small, they are of some importance, and so far have amounted to about

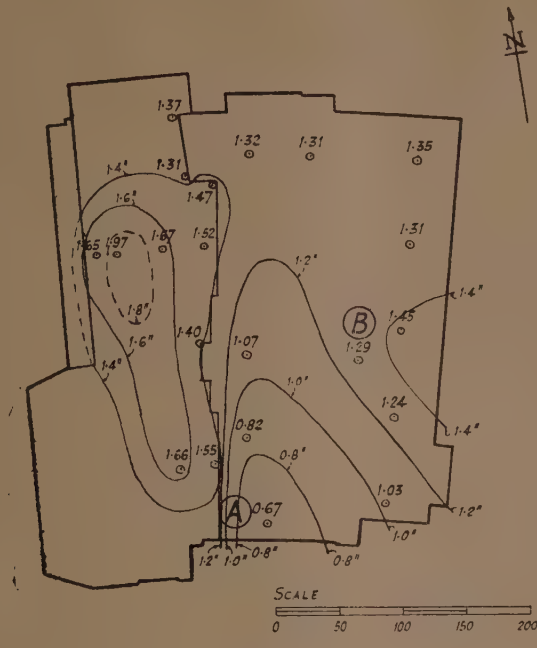


FIG. 7(a) OBSERVED TOTAL SETTLEMENT (IN.) (WINTER 1952/53)

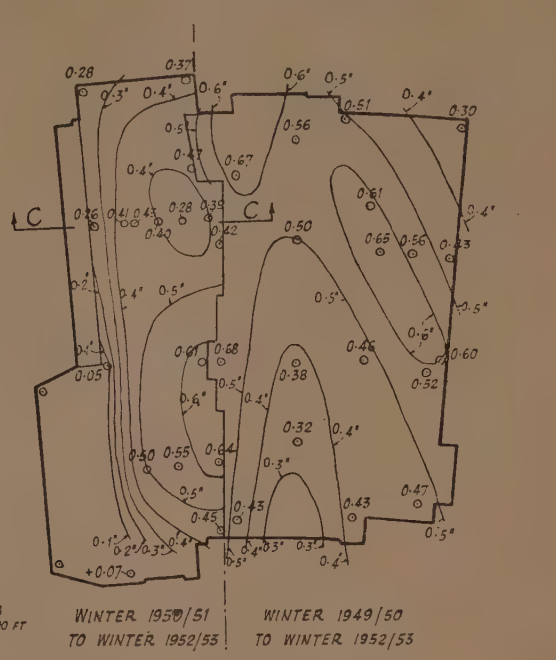


FIG 7(b) OBSERVED DIFFERENTIAL SETTLEMENT (IN.)

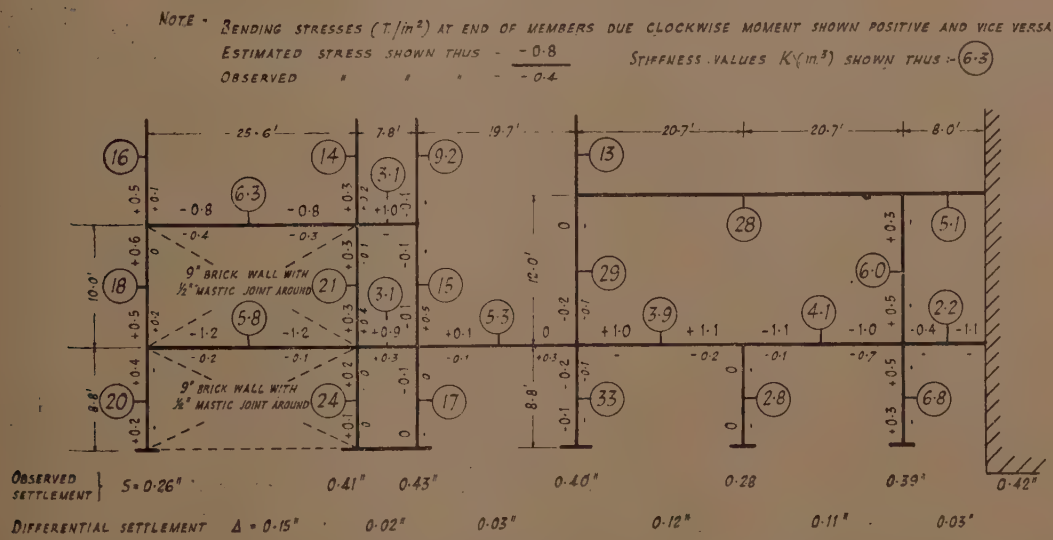

NOTE - BENDING STRESSES (T/in²) AT END OF MEMBERS DUE CLOCKWISE MOMENT SHOWN POSITIVE AND VICE VERSA
ESTIMATED STRESS SHOWN THUS - -0·8
OBSERVED " " " - -0·4
STIFFNESS VALUES K (in³) SHOWN THUS :- (6·3)
9" BRICK WALL WITH ½" MASTIC JOINT AROUND
OBSERVED SETTLEMENT S = 0·26" 0·41" 0·43" 0·40" 0·28 0·39" 0·42"
DIFFERENTIAL SETTLEMENT Δ = 0·15" 0·02" 0·03" 0·12" 0·11" 0·03"

Fig. 7 (c).—Settlement bending stresses in basement storeys of section C-C (Fig. 7 (b)) (Winter 1950/51 to Winter 1952/53)

one-quarter of the larger stresses due to normal structural loads.

4. Horizontal Movement and Tilting of Foundation and Effect on Superstructure

Where a foundation is subjected to a horizontal thrust H and bending moment M in addition to a vertical load, horizontal movement and tilting must be expected in addition to settlement. According to elastic theory[7] the stresses in the direction of a load on an isotropic half-space with effective modulus E'_s are independent of the load inclination. For practical purposes this is also sufficiently accurate for the displacements so that for a horizontal foundation the total movement S in the direction of a pressure p inclined at α to the vertical can, as a first approximation, be estimated from eqs. (7) to (12) ; the horizontal and vertical components of the movement are then $S.\ sin\ \alpha$ and $S.\ cos\ \alpha$,respectively. Similarly, for a vertical eccentric load and the customary assumption of linear contact pressure distribution, the average tilt of a foundation can be computed from elastic theory ; the analytical results obtained by the author may be illustrated by a square footing of width B (Fig. 8) when the tilt θ is sensibly directly proportional to eccentricity until the load approaches the footing edge

$$\text{and } \theta = \frac{3M}{E'_s B^3}, \text{ very nearly} \quad . \quad . \quad . \quad . \quad . \quad (18)$$

For an eccentric inclined load the above movements calculated separately are merely superimposed. The movements observed on model footings under eccentric

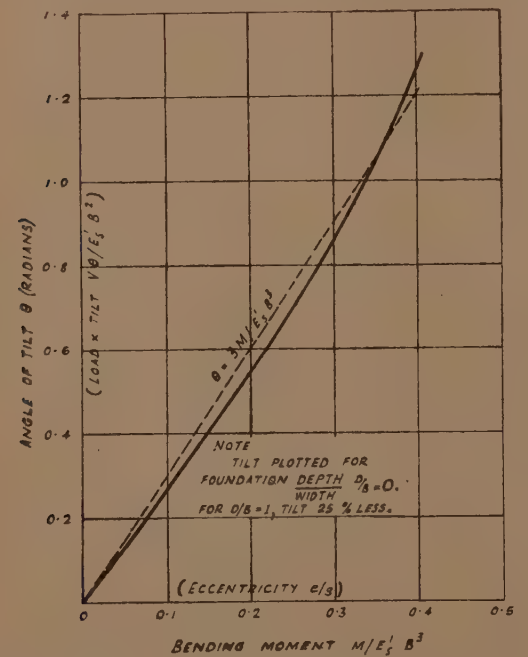


Fig. 8.—Tilting of square footing under eccentric load

and inclined load on clay[4] agreed reasonably well with the above analysis, but no field observations are available as a check.

Although horizontal and tilting movement can affect the stresses and stability of load bearing walls, it is especially important in arched and framed structures of single storey height. In multi-storey building frames the effect is greatest in the bottom storey and decreases rapidly with distance from the base. The bending moments and shearing forces induced by the movement can be estimated, in a similar manner to that proposed for differential settlement, by a structural analysis in conjunction with the deformation characteristics of the soil based on the above relationships. Such an analysis has been made for the lowest columns of building frames on square footings with " hinged " and " fixed " feet to determine the horizontal thrust H and bending moment M at footing level after movement (Appendix 3).

The results (Figs. 9a and b) show that the thrust H and moment M (expressed in terms of the customary values H_o and M_o, respectively, for no footing movement) decrease rapidly as the relative stiffness K'_r of the column ($E'I/h_1$) to that of the soil ($E'_s B^3$) increases, and as the footing width (B/h_1) increases. The decrease in bending moment is particularly marked because it also depends on the lateral movement as well as tilt of the footing; since K'_r can vary in practice within wide limits (about 0 to 10 or more), it will be seen that unless the footings are exceptionally small, the moment may be practically zero or even be reversed due to lateral movement changing the eccentricity on the base. The relative column stiffness at the head has generally comparatively little influence on the results but the beneficial effect of a tie between the feet is clearly shown (Figs. 9a and b). Knowing H and M the corresponding stresses in the superstructure follow from standard analysis. It follows that foundation movement, especially horizontal and tilting, can govern the maximum H, M and V which a column can resist at least at footing level in the absence of soil or structural failure ; and such movements may therefore affect the mode of failure and limit the collapse load of superstructures.

To study the interaction between simple frame structures and the soil, especially in relation to horizontal movements and tilting of the foundation, some loading tests were made on 4 in. high and 8 in. wide model portal frames with 1 in. square footings resting on clay and sand. To cover the limiting conditions the footings were either pads on the surface or blocks at 1 in. depth and covered with sand-paper to ensure full mobilisation of the shearing strength of the soil ; in some tests a stiff tie was provided at base level. The portals were made of annealed mild steel (lower yield stress = 16.2 t/in.^{2} from tensile tests) of either $\frac{1}{4}$ in. square (stiff frame) or $\frac{1}{4}$ in. wide by $\frac{1}{8}$ in. deep (slender frame) section throughout. The soil was soft remoulded London clay (cohesion = 2.4 lb./in.^{2}) or compact Hanri River sand (angle of internal friction = 45°). Control tests were made on centrally loaded footings (average results in Table 2) and simply supported beams, which gave a mean lower yield stress = 16.4 t/in.^{2} and modulus of elasticity = 12,000 t/in.^{2}.

The frames were loaded with a central point load on $\frac{1}{4}$ in. width of the beam. The load was applied by a proving ring at a slow and fairly constant rate of deformation until structural collapse or failure of the soil occurred. The vertical movement of the beam, centre and ends (= footing settlement) and the horizontal movement of the column feet (= footing displacement) were recorded by dial gauges ; the rotation of the footings was observed by mirrors (Fig. 10a). In some tests a vertical load was applied together with a horizontal load on the beam and both were increased to failure, the horizontal beam displacement being measured by an additional gauge. Typical portals after failure by vertical load and by combined vertical and horizontal loads are illustrated in Figs. 10 b and c. The main test results are summarised in Table 2, and typical relationships between applied load and observed movements are given in Figs. 11 and 12a and b for a stiff frame with

oil failure and slender frames with structural collapse nder vertical and combined loads.

In general, the load-settlement relation of the footings was sensibly independent of the stiffness of the portal and presence of a tie and was therefore similar to the relationship of the corresponding single footing control est. In frames under combined vertical and horizontal oads the leeward footing settled more than the wind-ward footing, roughly in proportion to their respective of these loads and was accompanied by some spread of the feet unless they were tied.

The rotation of the footings of the stiff frames without tie under vertical load or combined vertical and small horizontal loads was generally outwards (negative) at small loads and then inwards (positive) at greater loads due to lateral movement reversing the original footing bending moment ; the same condition held for the slender frames on the surface. When footing spread was

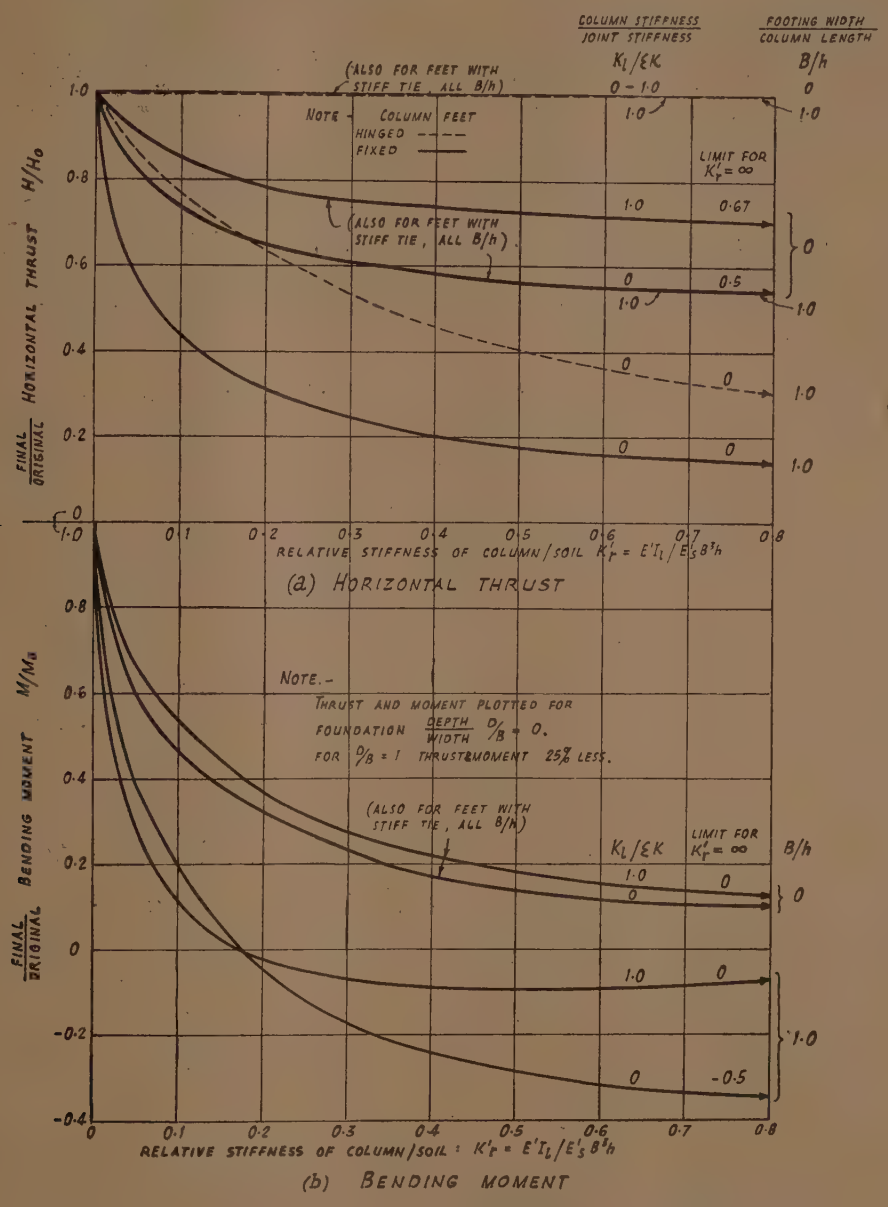

COLUMN STIFFNESS / JOINT STIFFNESS
FOOTING WIDTH / COLUMN LENGTH
(ALSO FOR FEET WITH STIFF TIE, ALL B/h)
NOTE - COLUMN FEET
HINGED
FIXED
LIMIT FOR K'r = ∞
FINAL / ORIGINAL HORIZONTAL THRUST H/H0
RELATIVE STIFFNESS OF COLUMN/SOIL K'r = E'IL / E's B³h
(a) HORIZONTAL THRUST
NOTE.-
THRUST AND MOMENT PLOTTED FOR FOUNDATION DEPTH / WIDTH D/B = 0.
FOR D/B = 1 THRUST&MOMENT 25% LESS.
FINAL / ORIGINAL BENDING MOMENT M/M0
(b) BENDING MOMENT

Fig. 9.—Effect of relative stiffness on horizontal thrust and bending moment at base of column on square footing

vertical loads. The horizontal and tilting movement of the footings of the stiff frame under a given load was less than that of a similar flexible frame, but the difference was not nearly as great as that of the flexural rigidities ; the movements of the stiff frame corresponded therefore to a much smaller horizontal thrust and bending moment at the footing compared with the flexible frame. The horizontal footing movement of frames subjected to vertical load or combined vertical and small horizontal loads was outwards (positive) unless prevented by a tie ; under greater horizontal loads the footing movement occurred in the direction considerably reduced as in the slender frames at depth, or was prevented by a tie, the footings tilted outwards due to moment reduction. The footing rotation under combined vertical and larger horizontal loads followed the overturning moment induced by the horizontal load. The foundation movements in clay and sand were of the same type and decreased with greater footing depth as would be expected. These experimental results give some qualitative support for the above theoretical analysis, which can be checked quantitatively for the frames on clay, as illustrated in Fig. 12a. The movements, which were estimated from equations (38) to

(40) (Appendix 3) using an effective modulus of the clay from compression tests as for immediate settlement (Section 2), are of the same order as those observed.

As was to be expected, all the tests with stiff portals exhibited soil failure because the estimated collapse

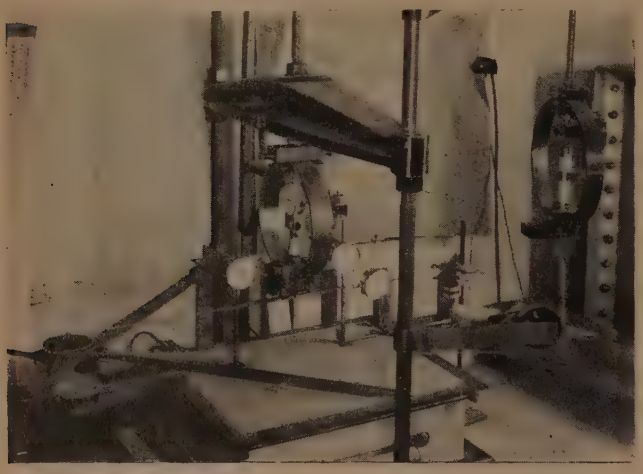


Fig. 10 (a).—Arrangement of loading test on model portal frame under combined vertical and horizontal loads

load of the frame was 144 lb. compared with an estimated bearing capacity of the soil from 10 to 70 lb., depending on the soil type and footing depth. The load at failure (Table 2) was practically the same as that of the corresponding control footings since the inclination and eccentricity of the load on the soil (i.e., horizontal thrust and bending moment) were very small on account of the large relative stiffness of frame to soil. Accordingly, the provision of a tie did not affect the failing load.

The slender portals failed generally by structural collapse. Two frames without ties on the surface, however, failed at a reduced bearing capacity of the soil because the small relative stiffness of the frame allowed a substantial inclination and eccentricity of the load on the soil to be developed. To induce a soil

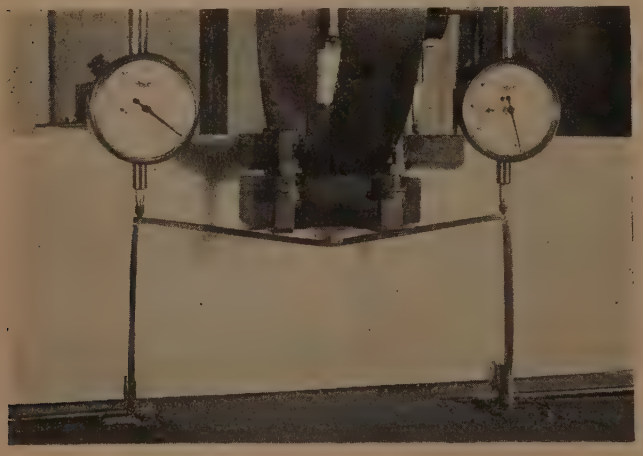


Fig. 10 (b).—Failure of slender portal frame without tie under vertical load on clay (structural collapse)

failure at depth, one of the frames (No. 17) on sand was not annealed. Failure of the other portals subjected to vertical load or combined vertical and small horizontal loads began with a plastic hinge at the centre of the beam and was, with a few exceptions (see below), followed by plastic hinges at the beam-column joints, usually at the head of the columns on account of the

effect of axial load ; no plastic hinges were formed a the base of the columns. According to plastic theory the centre of the beam is expected to reach full plasticit at a vertical load of 30 lb. for a frame with a fixed bas while at 36 lb. structural collapse should occur provide

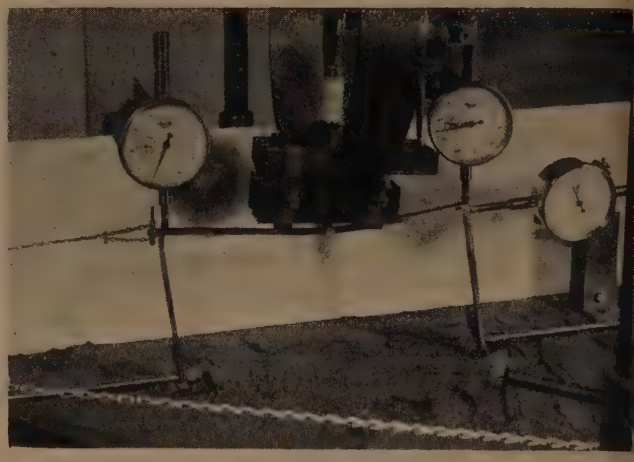


Fig. 10 (c).—Failure of slender portal frame without tie under combined vertical and horizontal loads on clay (structural collapse)

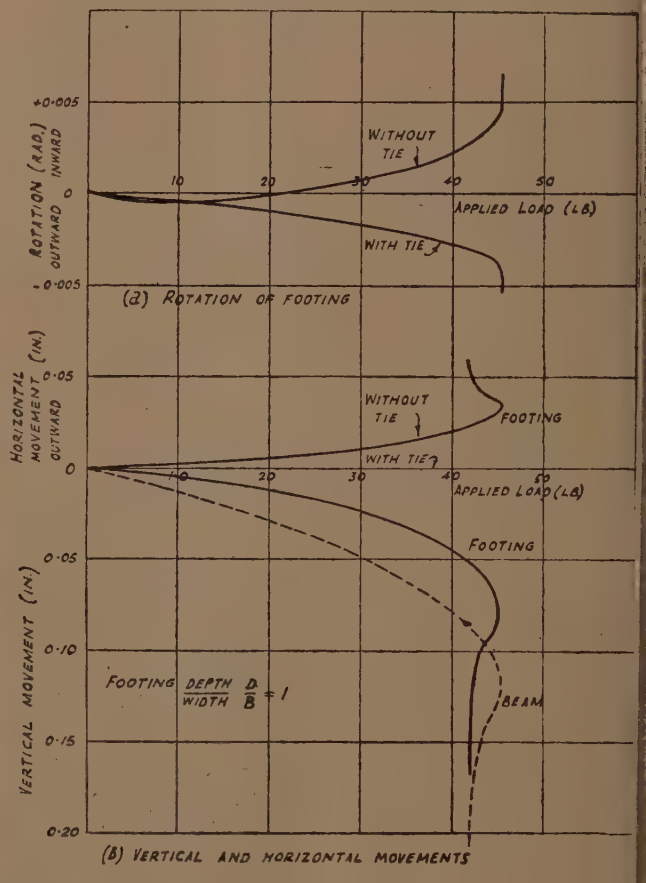


Fig. 11.—Typical results of loading tests on stiff portal frames under vertical load on sand (soil failure)

the horizontal load is less than one-half of the vertic load.

The observed failing loads of slender portals und vertical load (Table 2) are close to the estimates an somewhat greater due to some strain hardening at th plastic hinges for all frames with tie and for the fram on deep footings without tie on clay. For the fram

TABLE 2.—Results of Loading Tests on Portal Frames

(a) Footings on Clay

Frame No.	Frame Section	Footing Depth	Footing Restraint	Failing Load Beam	Failing Load Column	Failing Load Soil (total)	Mode of Failure	Bearing Capacity of Control Footing (mean)	Remarks
	(in.)	(in.)		(lb.)	(lb.)	(lb.)		(lb.)	
			(i) Vertical Load						
21	¼ × ¼	0	—	—	—	31.2	Soil	34.5	
22	,,	,,	Tie	—	—	33.0	,,	,,	
23	,,	1	—	—	—	65.0	,,	64.0	
29	,,	,,	—	—	—	61.8	,,	,,	
24	,,	,,	Tie	—	—	56.0	,,	,,	
25	¼ × ⅛	0	—	26.7	—	28.2	Frame (1)	34.5	
26	,,	,,	Tie	30.8	36.7	37.1	,, (2)	,,	
27	,,	1	—	30.0	35.9	36.7	,, (2)	64.0	
28	,,	,,	Tie	32.4	37.2	38.2	,, (2)	,,	
			(ii) Vertical and Horizontal Loads						
30	¼ × ¼	0	—	—	—	17.5	Soil	17.5	Vertical Load
						6.1		6.8	Horizontal ,,
31	,,	1	—	—	—	30.8	,,	30.6	V
						19.4		20.0	H
33	,,	,,	Tie	—	—	31.0	,,	,,	V
						20.5			H
34	¼ × ⅛	0	—	—	—	10.1	,,	10.1	V
						6.7		5.2	H
35	,,	,,	—	22.4	—	23.2	Frame (1)	32.0	V
				0.5		0.5		0.8	H
36	,,	1	—	24.8	21.8	25.6	,, (3)	32.5	V
				9.0	4.7	9.0		11.4	H
39	,,	,,	Tie	24.3	23.2	25.2	,, (3)	33.0	V
				8.4	7.3	8.4		11.0	H
37	,,	,,	—	—	5.5	6.3	,, (4)	11.5	V
					16.5	16.5		30.0	H

For observed modes of structural collapse, see Note below Table 2b.

(b) Footings on Sand

Frame No.	Frame Section	Footing Depth	Footing Restraint	Failing Load Beam	Failing Load Column	Failing Load Soil (total)	Mode of Failure	Bearing Capacity of Control Footing (mean)	Remarks
	(in.)	(in.)		(lb.)	(lb.)	(lb.)		(lb.)	
			(i) Vertical Load						
5	¼ × ¼	0	—	—	—	10.5	Soil	13.0	
12	,,	,,	—	—	—	12.4	,,	,,	
13	,,	,,	Tie	—	—	12.8	,,	,,	
6	,,	1	—	—	—	48.6	,,	46.5	
10	,,	,,	—	—	—	46.2	,,	,,	
9	,,	,,	Tie	—	—	43.5	,,	,,	
11	,,	,,	,,	—	—	55.8	,,	,,	
20	¼ × ⅛	0	—	—	—	8.4	,,	13.0	
17	,,	1	—	—	—	46.8	,,	46.5	Not Annealed
18	,,	,,	—	31.4	—	33.4	Frame (1)	,,	
19	,,	,,	Tie	33.8	35.6	36.5	,, (2)	,,	
			(ii) Vertical and Horizontal Loads						
7	¼ × ¼	0	—	—	—	5.5	Soil	5.5	Vertical Load
						2.2		2.5	Horizontal ,,
16	,,	1	—	—	—	26.5	,,	26.4	V
						7.5		6.0	H

Note : Observed modes of structural collapse denoted thus :—

Frame (1) : Plastic hinge at centre of beam only.
Frame (2) : Plastic hinges at beam centre and beam-column joints.
Frame (3) : Plastic hinges at beam centre and leeward beam-column joint.
Frame (4) : Plastic hinge at leeward beam-column joint only.

without tie on the surface of clay and at depth in sand, however, the soil resistance to lateral and rotational foundation movement was insufficient to permit plastic hinges to develop at the beam-column joints and thus reduced the collapse load by up to about 40 per cent. Similarly, for all slender frames at depth in clay under combined vertical and horizontal loads, when failure began with a plastic hinge at the leeward beam-column joint and was sometimes followed by a hinge at the beam centre, no plastic hinges were developed at the windward joint nor at the column feet even when they were tied ; the corresponding collapse loads were therefore much less than estimated on the basis of no footing movement. It may be noted that for the type of structure investigated the maximum theoretical reduction in the collapse load due to foundation movements is 50 per cent., which is however unlikely to arise in practice. It is hoped that field observations are made on full-sized structures to determine the lateral and rotational movement of the foundation as well as the settlement and to study their effect on the behaviour of different types of superstructures.

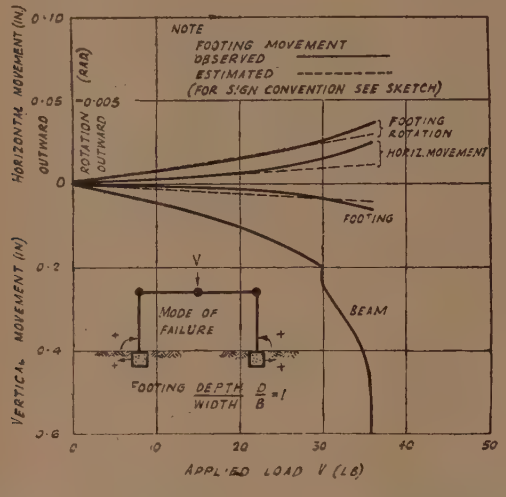


Fig. 12 (a).—Typical results of loading test on slender portal frame without tie under vertical load on clay (structural collapse)

Conclusion

Rational foundation design requires an assessment of the stability of the soil and the movement of the foundation and its effect on the behaviour of the superstructure. The ultimate bearing capacity of shallow and deep foundations under vertical load can be estimated from soil and foundation characteristics under most conditions in practice. Eccentricity and inclination of the load reduce the bearing capacity of the soil and may limit the maximum bending moment and shearing force which can be resisted by a column at base level.

The settlement of foundations is more difficult to estimate although approximate relationships have been suggested for preliminary estimates of the total settlement of shallow foundations ; a more accurate prediction for shallow and deep foundations can only be made from a detailed site investigation. The differential settlement depends on the layout and rigidity of the whole structure in addition to the soil and foundation characteristics. Approximate methods have been developed to estimate the stiffness of structures and to relate it to the deformation characteristics of the soil. This analysis shows that the differential settlement decreases rapidly as the relative stiffness of structure to soil increases, while correspondingly greater bending moments and shearing forces are induced in the structure. The results obtained can be applied directly to the design of uniformly loaded rafts and centrally loaded strip footings.

Recent observations on the behaviour of brick walls and cladding to frames have been analysed to suggest approximate safe limits for the deformation and stresses from which the required factor of safety on the ultimate bearing capacity of foundations can be assessed. A study of the movements of a large cladded framed building on a raft on London clay illustrates the effect of structural stiffness on the settlement distribution. While the

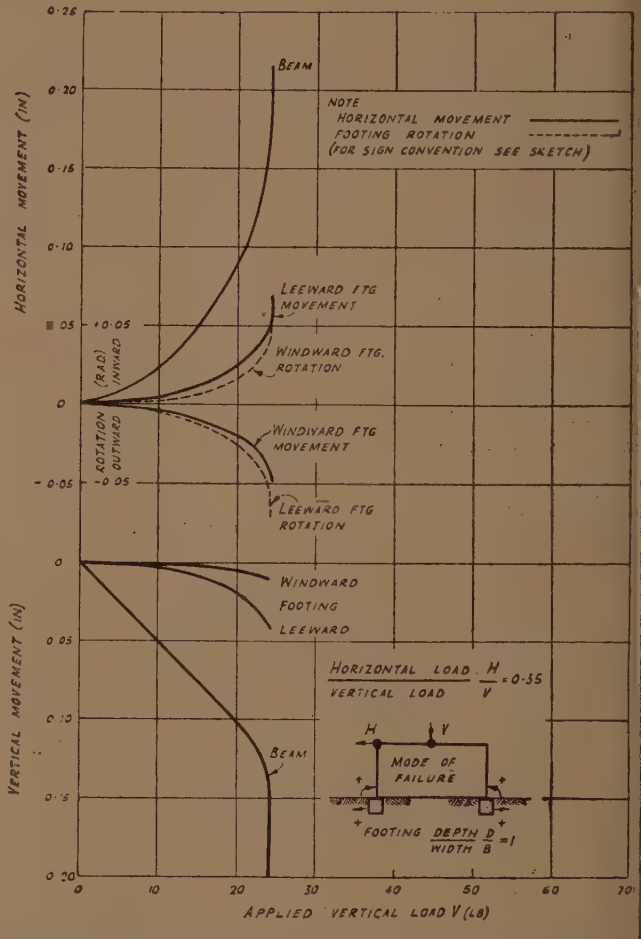


Fig. 12 (b).—Typical results of loading test on slender frame with tie under combined vertical and horizontal loads on clay (structural collapse)

estimated and observed total settlements compared reasonably well, the estimated stresses in the steelwork assuming bare and rigidly connected members were considerably less than those observed on account of the cladding of the structure.

Although settlement is generally the most important type of foundation movement, horizontal movement and tilting of foundations has a considerable influence on the behaviour of walls, single-storey structures and the lower parts of multi-storey buildings. An analysis of this problem shows that as the relative stiffness of the lowest columns to the soil increases, the horizontal and tilting movements increase, while the corresponding horizontal thrust and bending moment at the column base decrease rapidly. Some support for this analysis has been obtained from the results of loading tests on

odel portal frames on clay and sand, which also indicate ıe beneficial effect of a tie between the feet.

Acknowledgement

The author is indebted to his colleagues for assistance the settlement observations at Whitehall Gardens and carrying out the model portal tests. He is parcularly grateful to Dr. L. F. Cooling for his interest and ıcouragement and to Dr. R. H. Wood for stimulating scussions; Dr. Wood was also responsible for the easurement of the stresses at Whitehall Gardens.

The work was carried out as part of the research rogramme of the Building Research Board of the epartment of Scientific and Industrial Research, and ıe paper is published by permission of the Director of uilding Research.

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Appendix

Stiffness of Multi-Storey Building Frames

(1) *Open (bare or encased) Frame*

A building frame of length L with maximum (central) ifferential settlement $\triangle max$ and central points of ontraflexure in the columns (Section 2) can be split into ngle bay component frames of beam length l and verage storey height $h = (h_1 + h_u)/2$ with hinged ends Fig. 13a). To represent an interior bay of the main frame, the component frame has a beam stiffness K_b and columns stiffnesses $K_l/2$ and $K_u/2$.

where $K_b = I_b/l$, $K_l = I_l/h_l$ and $K_u = I_u/h_u$ are the stiffnesses of the members of the main frame (or equivalent stiffnesses in a semi-rigid frame).

Then for a differential settlement δ of the supports of the component frame, it can be shown that the bending moment at the end of the beam is

$$M_b = 6\,E'K_b\left(\frac{K_l + K_u}{K_b+K_l+K_u}\right)\frac{\delta}{l} \quad . \quad . \quad . \quad (20)$$

where E' = effective modulus of elasticity to allow for elastic and creep deformations.

Consider a beam line of length L and flexural rigidity $E'I_b$ of the main frame subjected to opposite external bending moments M_o (Fig. 13a). Then the central deflection

$$\triangle max = \triangle_o - \triangle_b \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (21)$$

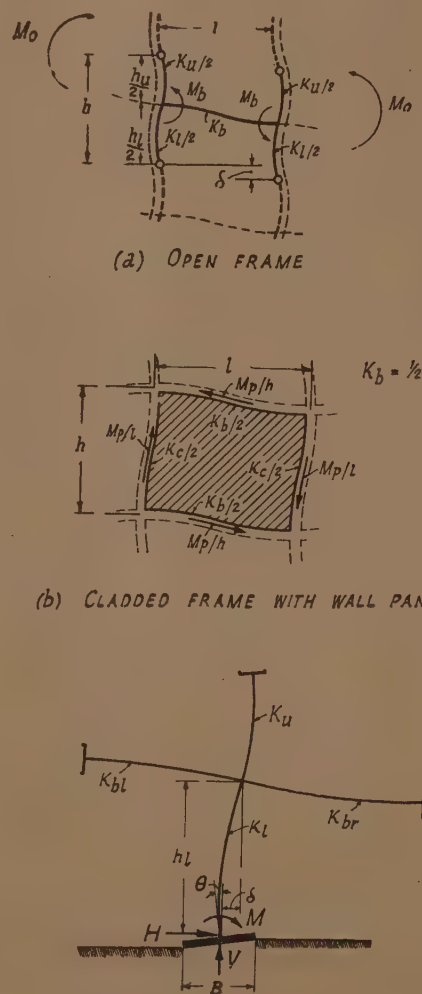

Fig. 13.—Component frames of interior bays of multi-storey building frames

where $\triangle_o$ = downward deflection due to M_o

$$= \frac{M_oL^2}{8E'I_b} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (22)$$

and $\triangle_b$ = upward deflection due to 2 M_b at every (internal and external) joint

$$= \frac{M_b\ L^3}{12\ E'I_b l} \quad \ldots\ldots\ldots\ldots \quad (23)$$

when $\delta = 2l\ \triangle max/L$ $\ldots\ldots\ldots\ldots$ (24)

Substituting eqs. (20) and (22) to (24) into (21)

$$\triangle max = \frac{M_o L^2/8\ E'I_b}{1 + \left(\frac{K_1 + K_u}{K_b + K_1 + K_u}\right)\frac{L^2}{l^2}} \quad \ldots \quad (25)$$

$$= \frac{M_o L^2}{8E'I'_b} \quad \ldots\ldots\ldots\ldots \quad (26)$$

where $E'I'_b$ = effective flexural rigidity of the beam line or storey.
Hence from eqs. (25) and (26)

$$E'I'_b = E'I_b \left[1 + \left(\frac{K_1 + K_u}{K_b + K_1 + K_u}\right)\frac{L^2}{l^2}\right] \quad (27)$$

(b) *Cladded Frame with Panels*

An interior bay of a framework transmitting only shearing stresses to solid infilling panels can be represented by a component frame with average beam and column stiffnesses of $K_b/2$ and $K_c/2$, respectively, where $K_c = I_c/h$ (Fig. 13b). For a differential settlement δ, the moment at end of the beam is $M_b/2$, where M_b is given by eq. (20) ; further, the total bending moment from the shearing stresses on the panel of thickness b, shear modulus G and Poisson's ratio $1/m$ is

$$M_p = G\ bh\ \delta = \frac{6E'_p I_p \delta}{(1 + 1/m)h^2} \quad \ldots\ldots\ldots \quad (28)$$

where $E'_p I_p$ = flexural rigidity of panel
$= E'_p bh^3/12$.
For a beam line with M_o as before

$$\triangle max = \triangle_o - \triangle_b + \triangle_p \quad \ldots\ldots\ldots \quad (29)$$

where $\triangle_p$ = upward deflection due to M_p at every panel

$$= \frac{M_p L^3}{24\ E'I_b l} \quad \ldots\ldots\ldots\ldots \quad (30)$$

Substituting eqs. (28), (30) and the previous results into (29) and putting $1/m = 0$, the total effective flexural rigidity of the storey is

$$E'I'_b + \frac{E'_p I_p L^2}{2h^2} \quad \ldots\ldots\ldots\ldots \quad (31)$$

where the first term represents the effect of the framework (eq. 27) and the last term the effect of the panels.

2. Bending Moments and Shearing Forces in Rigid Strip Footing

(a) *Uniformly Distributed Load*

The contact pressure p_x at a distance x from the centre of a uniformly loaded (pressure = p) rigid strip footing of width B on an elastic solid is[11]

$$p_x = \frac{2p}{\pi\sqrt{(1 - (2x/B)^2)}} \quad \ldots\ldots \quad (3$$

∴ Transverse bending moment at distance a fro centre is

$$M_a = \int_a^{B/2} p_x\ (x - a)\ dx - \frac{p}{2}\left(\frac{B}{2} - a\right)^2$$

$$= p\ B^2 \left[\frac{1}{2\pi}\sqrt{(1 - (2a/B)^2)} + \frac{a}{B}\left\{\frac{\sin^{-1}(2a/B)}{\pi} - \frac{1}{2}\right\} - \frac{1}{8}\left(1 - \frac{2a^2}{B}\right)\right] \quad (3$$

with a maximum $M_{max} = \frac{pB^2}{29.2}$ at the centre.

Further, the transverse shearing force at a is

$$V_a = \int_a^{B/2} p_x\,dx - p\ \left(\frac{B}{2} - a\right)$$

$$= pB\left[\frac{a}{B} - \frac{\sin^{-1}(2a/B)}{\pi}\right] \quad \ldots\ldots \quad (3$$

with $V_{max} = 0.105\ pB$ at $a = 0.386B$.

The longitudinal bending moments and shearing forc are zero.

(b) *Central Line Load*

For a footing as before but with a central line load per unit length

$$p_x = \frac{2P}{\pi B\sqrt{(1 - (2x/B)^2)}} \quad \text{from eq. (3}$$

$$\therefore M_a = \int_a^{B/2} p_x\ (x - a)\ dx$$

$$= P\ B\left[\frac{1}{2\pi}\sqrt{(1 - (2a/B)^2)} + \frac{a}{B}\left\{\frac{\sin^{-1}(2a/B)}{\pi} - \frac{1}{2}\right\}\right] \quad (3$$

with $M_{max} = \frac{PB}{2\pi}$ at centre.

Similarly, $V_a = \int_a^{B/2} p_x\,dx$

$$= P\left[\frac{1}{2} - \frac{\sin^{-1}(2a/B)}{\pi}\right] \quad \ldots\ldots \quad (36)$$

with $V_{max} = \frac{P}{2}$ at centre.

The longitudinal bending moments and shearing forces are again zero. The transverse bending moments and shearing forces for uniformly distributed and line loads are shown in Fig. 5.

3. Lateral Movement and Tilting of Footings of Building Frames

a) *"Fixed" Column Feet*

Let Fig. 13c represent an internal beam-column joint of the bottom storey of height h_1 of a multi-storey frame with the (actual or equivalent) stiffnesses of the members as shown and a horizontal thrust H_o, bending moment M_o and vertical load V_o on a horizontal and fixed column base. If now the ends remote from the joint of all members are fixed except the base, which is displaced horizontally by δ and rotated by θ as indicated, it can be shown by the slope-deflection method (taking clockwise moments and rotations as positive) that the resulting horizontal thrust H and bending moment M at the base of the component frame (Fig. 13c) can be expressed by

$$\left.\begin{aligned} H &= H_o - \frac{2E'K_1}{h}\left\{\theta+(\theta+3\delta/h_1)(2 - 3K_1/2\Sigma K)\right\} \\ \text{and } M &= M_o - 2E'K_1\left\{\theta+(\theta+3\delta/h_1)(1 - K_1/2\Sigma K)\right\} \end{aligned}\right\} \quad (37)$$

where $\Sigma K = K_{b1}+K_{br}+K_1+K_u$
and other symbols as before.
The resulting vertical load V affects the settlement of the footing only and need not be considered now.

For a square footing of width B on a soil with constant effective modulus E'_s it was indicated (Section 4) that approximately

$$\delta = \frac{H}{E'_s B} \quad \ldots\ldots \quad (38)$$

$$\text{and } \theta = \frac{3M}{E'_s B^3} \quad \ldots\ldots \quad (39)$$

Substituting these relationships into eq. (37)

$$\left.\begin{aligned} H &= H_o - \frac{6K'_r}{h_1}\left\{M+(M+B^2H/h_1)(2 - 3K_1/2\Sigma K)\right\} \\ \text{and } M &= M_0 - 6K'_r\left\{M+(M+B^2H/h_1)(1 - K_1/2\Sigma K)\right\} \end{aligned}\right\} \quad (40)$$

where K'_r = relative stiffness of column to soil

$$= \frac{E'I_1}{E'_s B^3 h_1}$$

from which H and M can be determined in any given case.
For a rigid soil with $E'_s = \infty$, from eq. (40)
$H = H_o$ and $M = M_o$ as in customary analysis.
If between the column feet a flexible tie is provided of length l, area A_t and modulus of elasticity E_t, then extension of tie under force H_t in tie for symmetrical loading and layout, is

$$2\delta = \frac{H_t l}{A_t E_t}$$

$$= \frac{2(H - H_t)}{E'_s B} \text{ for footings from eq. (38).}$$

Eliminating H_t,

$$\delta = \frac{H}{E'_s B\left(1+\frac{2A_tE_t}{BlE'_s}\right)} \quad \ldots\ldots \quad (41)$$

Substituting eqs. (39) and (41) into (37), the term B^2H/hl in eq. (40) is now replaced by

$$\frac{B^2H}{h_1\left(1+\frac{2A_tE_t}{BlE'_s}\right)} \quad \ldots\ldots \quad (42)$$

For an inextensible tie ($E_t = \infty$), $B^2H/h_1 = 0$.

(b) *"Hinged" Column Feet*

If in the above frame (Fig. 13c) hinges are provided at the column feet ($M_o = 0$) so that after horizontal displacement δ the column base remains horizontal ($M = 0$), it can be shown that the resulting horizontal thrust

$$H = H_o - \frac{3E'K_1\,\delta}{h^2_1}\left(1 - \frac{3K_1}{4\Sigma K'}\right) \quad \ldots\ldots \quad (43)$$

where $\Sigma K' = K_{b1}+K_{br}+3K_1/4+K_u$
and other symbols as before.
Substituting eq. (38) into (43)

$$H = \frac{H_o}{1+\frac{3K'_rB^2}{h^2_1}\left(1 - \frac{3K_1}{4\Sigma K^1}\right)} \quad \ldots\ldots \quad (44)$$

with symbols as before.
For a tie between the feet as before, from eqs. (41) and (43)

$$H = \frac{H_o}{1+\frac{3K'_rB^2}{h^2_1}\left(1 - \frac{3K_1}{4\Sigma K'}\right)\div\left(1+\frac{2A_tE_t}{BlE'_s}\right)} \quad (45)$$

For a rigid soil or inextensible tie, $H = H_o$ as in customary analysis. The horizontal thrusts and bending moments for "fixed" and "hinged" column feet are shown in Figs. 9a and b.

Discussion

The Literature Committee would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be received by September 1st, 1953.

Electricity Generating Stations

Discussion on Mr. B. A. E. Hiley's Paper*

Correction

On page 11 of the January issue of "The Structural Engineer," column 1, line 2, the figure of approximately 1,000,000 kW. should be half a million. A consequential alteration is necessary in line 3.

Mr. HILEY, in presenting the paper, showed many illustrations of the installations described, together with a coloured film showing the progress of the work at Uskmouth, which included aerial views of the station.

The PRESIDENT, proposing a hearty vote of thanks to Mr. Hiley for his most interesting paper, said the meeting had appreciated the additional explanations he had given. The historical references were of great interest and added to the value of the paper. He felt that he would like to know much more about certain parts of the work, but he realised that that would have meant at least another paper, as Mr. Hiley had covered a wide field.

In a comment on the illustrations of the Brighton Power Station (Fig. 2), the President said he did not know whether, at the time it was constructed, there was a shortage of wrought iron or steel, but there certainly did not appear to be much waste of material there.

Major C. D. C. BRAINE (Member), referred first to Mr. Hiley's reference to "sewage gas stations," and said that, as they might become increasingly important in the future, a word about them to augment his remarks might not be out of place.

The term "sewage gas" was hardly correct, and he suggested that "sludge gas" was a better one.

In regard to the author's reference to "the late Mr. D. M. Watson, of the Birmingham Tame and Rea District Drainage Board," Major Braine was glad to say that his friend Mr. David Watson was still active and very much alive. The reference was, of course, to his father, Mr. J. D. Watson, a Past-President of the Institution of Civil Engineers. The son, Mr. D. M. Watson, referred to in the paper, was now Vice-President of the Institution of Civil Engineers.

The father and son were responsible for the West Middlesex Main Drainage Scheme, which included the biggest installation in Europe using sludge gas, and one of the biggest in the world. It used not far short of 1½ million cubic feet of gas per day. As many would know, sludge gas contained about 70 per cent. methane and had a calorific value of about 650 B.Th.U. per cubic foot of gas. The loss to the community by not making use of that valuable by-product of sewage could be considerable. For instance, if the sludge from London to-day were to be digested and the gas collected, it would produce gas equivalent to about 55,000 tons of coal per annum. It had long been incredible to him that in these days of fuel shortage that vast amount of sludge was still being barged to sea. One heard of bread bein cast upon the waters, but here we were casting th equivalent of 55,000 tons of coal per annum into the sea To do that, 1,500-ton sludge ships made about 1,00 trips per annum, and the fuel used by them must als be taken into account. Manchester and Glasgow als barged most of their sludge to sea, and even at Birming ham, where the process of sludge digestion was developec only partial use was made of the bulk of the sludg produced there, although plans were afoot to rectif that situation.

An example of what was involved was to be seen i the design of a large regional drainage scheme, wit which Major Braine had been concerned, for a grou of towns in the Near East. Had it not been for currency crisis, the construction of that scheme migh have been started already. The population which i was assumed would be connected eventually to th sewers of the Region had been taken for present purpose as 1,100,000, and the gas from the sludge digestio process would supply almost all the fuel for the powe required at the purification works and for pumping. Th ultimate installed horse-power in the power house woul be about 10,000 b.h.p.

Major Braine showed three slides to give some ide of the magnitude of the plant involved for a populatio of just over one million.

Slide 1 showed the general layout of the power house All the diesels were 680 b.h.p. dual fuel engines, som driving blowers, which together could compress a total o 130,000,000 cubic feet of free air per day, and som driving generators. Waste heat was used for sludg digestion. The pumps illustrated were capable of liftin 150 million gallons per day against a head of approxi mately 65 ft. The building was roughly 300 ft. lon and 70 ft. wide. The height of the ridge from operatin floor level was 45 ft. The cooling water required fo the plant was obtained from the purified effluent dis charged from the purification works.

Slide 2 showed a longitudinal section through th building. The caisson at one end housed the wet wel and pumps. It was about 133 ft. long and 40 ft. wide and its cutting edge was about 87 ft. below main floo level. The pump motors were located on the floo beneath the generators.

The air for the blowers was drawn in through louvre at one end, travelled along the length of the buildin passed down into the wet well and the dry well and wa then drawn up through a duct into an electrostati precipitator and thence along the main pipe gallery t the blower inlets. In that way it was hoped that som of the ventilation problems that arose in a hot climat would be solved satisfactorily.

Slide 3 showed cross-sections through the building an through the caisson. The earth bank that surrounde the building was provided in lieu of a blast wall.

As Mr. Hiley had said, sewage works were small powe producers, but they were exceedingly complex an

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**Read before a meeting of the Institution of Structural Engineers on Thursday, January 22nd, 1953. Mr. E. Granter, B.Sc., M.I.C.E., M.I.Struct.E. (President) in the Chair. Published in* THE STRUCTURAL ENGINEER, *Vol.* XXXI, *p.* 9. (*Jan.*, 1953.)

emanded from their designers an extraordinary variety f civil, mechanical and electrical experience. Although ne buildings Major Braine had illustrated were dwarfs, hen compared with the Uskmouth power station escribed in the paper, the structural problems involved ere quite sufficient to satisfy the most avid student f the Institution.

The electrical and mechanical side of such installations as extremely complicated, by reason of the various uties called for. One question which had to be decided as whether to adopt A.C. or D.C. In the two biggest uch stations with which he had been concerned—West Middlesex and the one in the Middle East—D.C. was hosen.

The likelihood, he concluded, was that a number of nstallations of the type he had illustrated would be alled for in this country before long.

Mr. F. M. Bowen (Associate Member of Council), ongratulated Mr. Hiley on the paper, and thanked him lso for his concise and interesting introduction. It as a very good thing, he said, that on occasions a istorical review of the subject under discussion should e given. In Mr. Hiley's paper the development of ower stations was illustrated clearly, including the remendous impetus which had been given by Parson's levelopments.

The development of the structural side appeared generally to have kept pace, but Mr. Bowen admitted that by comparison with the very pleasing Hams Hall "B" power station interior the engine house of which was shown in Fig. 6, he was a little disappointed with that of the Uskmouth station. Apart from the appearance he had gathered the latter was all steel framed ; yet n the engine house, illustrated in Fig. 11, the columns appeared to be cased.

Mr. Bowen suggested it would be desirable, if practicable, to introduce in the written discussion, an illustration of the crane girder expansion joint, for it was of considerable interest at the present time.

In regard to the special piling, he asked whether the steel casing was driven right down to practically the full length of the shaft, or whether it was driven only to the top of the marl, the shaft being then driven the remainder of the way. This piling was of special nterest to him because his firm working quite independently in the same district on another large job, had arrived at a similar solution under much the same conditions and for certain additional reasons, at about he same time.

Finally, in a reference to the delightful coloured photographs which Mr. Hiley had shown, he felt it must be agreed, in fairness to those associated with the ob on site, that the general elevation conveyed a totally erroneous impression of the nature of the average weather that was usually experienced in that part of he country.

Mr. L. E. Kent (Hon. Secretary), added his tribute o Mr. Hiley for an exceptionally interesting paper and or the very competent way in which he had put it over.

Referring to the expansion joints, he said there were wo in an approximate length of 900 ft., making about 300 ft. per bay. He asked whether there was a complete break made in the framework at those joints, and whether the joint was taken right through the brick covering of the structure to the outside.

In that connection he asked also whether, in the ntermediate lengths of 300 ft., only the one joint was made in the brick casing. He was thinking of relative expansion between the steel framework and the brick casing of the station.

The roof girders in the turbine house were designed to lift the transporter crane, and those in the boiler house were designed to lift the boiler drums. Mr. Kent could well appreciate that in the turbine house it was not possible, without going to a great deal of expense, to provide any sort of platform from which the crane could be erected, and consequently it would be economic to design the roof girders to lift the crane. But in the boiler house there was a structure on which the boilers were supported, and it would appear to be more economic to use that structure to help in erecting the drums.

In a particular case he had been asked to design the roof girders over a boiler house, which were 120 ft. above the ground, to lift the drums. The drums were rather heavier than those at Uskmouth, and he had declined to design the roof girders as requested because he had felt that to do so would be uneconomical, inasmuch as they would have had to be designed for a load which they would carry for a very short time, which load was much greater than that which they would sustain for the remainder of their working lives. The method finally adopted was to erect the boiler staging to within about 20 ft. of the roof ; a temporary staging was then erected through the roof girders and a further structure placed on top. The boiler drums were then lifted through the structure, with the consequence that the lattice girders over the boiler house were lighter than they would otherwise have been.

Mr. Avi Joshi (Graduate), paying his tribute to Mr. Hiley for an interesting paper, said it was a great pleasure to hear of generating stations not merely as structural masses, but as truly large engineering undertakings.

From the various illustrations he had been impressed by the grandeur of the Uskmouth chimneys. The author had suggested the very great economy of ferro-concrete over brick chimneys, and Mr. Joshi asked whether the brick chimneys illustrated in the slide showing comparisons were designed on the basis of the neutral axis falling within the section.

It was usual, he continued, to design a brick chimney so that under the worst wind conditions the entire section was under compression. The neutral axis in that case was outside the section. The limiting case for such a design would be when the compressive stress varied from zero to a safe maximum. The neutral axis was then tangential to the section. Any further increase in the bending stress would introduce tension. Assuming brickwork incapable of withstanding tension, the stress diagram would modify so as to produce an increase of compressive stress.

He asked if it were possible, with a brick chimney designed on the latter basis, and where the thickness of the shaft was increased only when the safe maximum stress was attained, that it would compare favourably with reinforced concrete.

In reinforced concrete chimneys the economical stress in the steel was often limited (about 5,000 to 12,000 lb./sq. in.). The use of a higher tensile stress shifted the neutral axis so that the area under compression was greatly reduced. Increases in steel stress due to temperature might not be very great in some power station chimneys, particularly where they were lined to full height and where the flue gases were of fairly low temperature. A low working stress in steel, on account

of limiting compression on concrete, did not appear to be a very satisfactory solution. In such cases it seemed possible that prestressed concrete might be used so that the full section would be available to resist compressive stresses. The question of creep and stress relaxation, etc., at higher temperatures, however, should be considered. He had been pleased to hear Mr. Hiley say that the Uskmouth chimney was in prestressed concrete.

At Uskmouth a 175-ton crane was used on gantry girders. That crane was required during the installation of the generators, but very seldom later. Mr. Joshi asked whether an economical solution would be to use a special lifting gear for those rare events. The lifting gear could be in the form of a portal crane running on rails at ground level, a light gantry crane being provided for ordinary work. Possibly a few hundred tons of steel would be required to support the conventional 175-ton gantry crane on tall columns.

Lastly, he commented on Mr. Hiley's remark that "There can be very little that is new in the structural design of a modern power station." He suggested that in most structures the real economy in design lay, not just in the use of higher stresses, but also in a more rational structural arrangement, the proper choice of material and methods of construction.

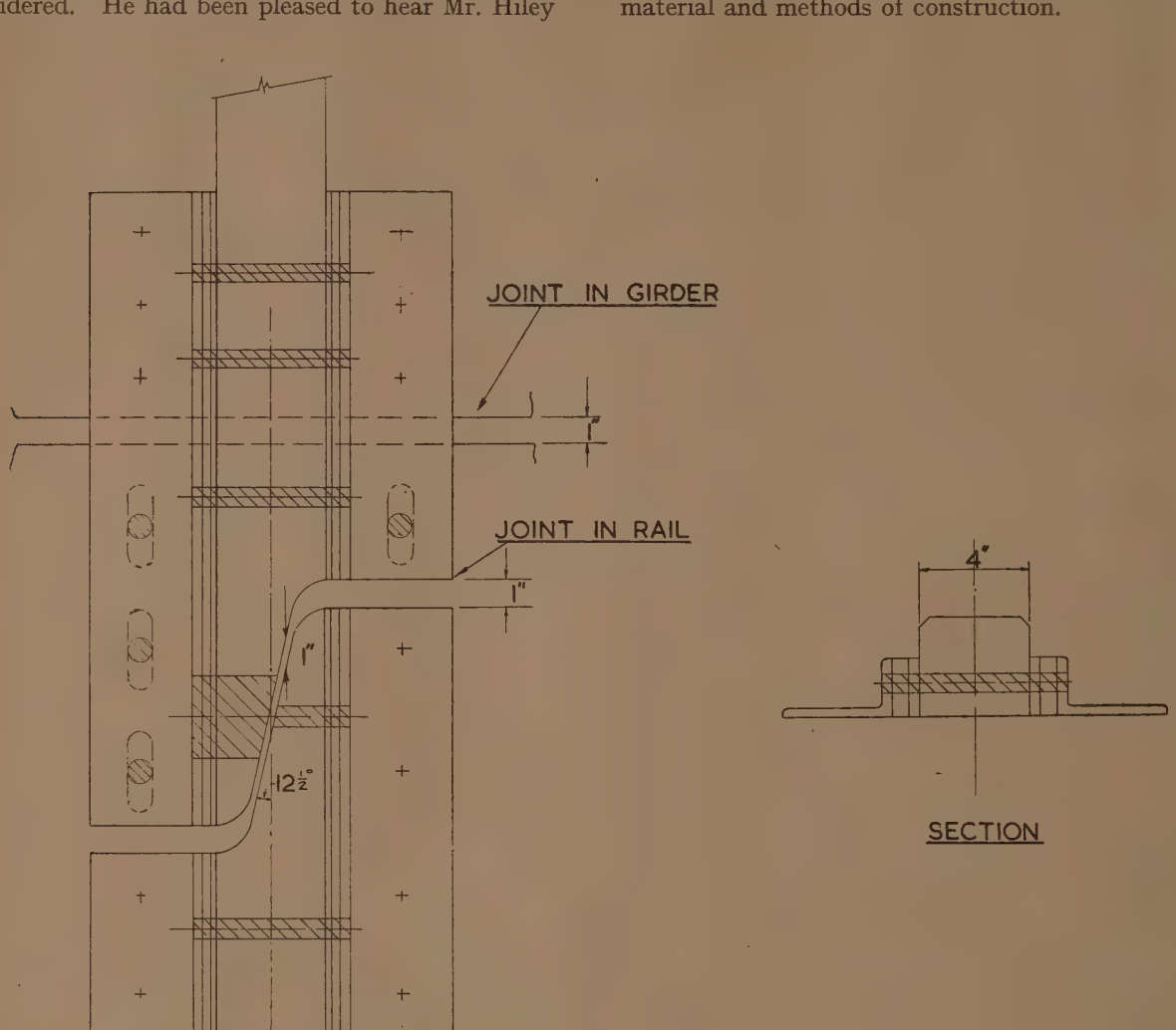


Fig. 1

In conventional designs of power stations the steel work was used to support plant and cladding. Where the latter was heavy, like brick partitions, the steel framework needed to support it would be substantial. He asked if it were possible that a covering shell would provide adequate shelter. Such a form would not only save steel, but it would make the erection of the structure independent of power station plant and would also allow unrestricted space for future modifications. The shell could be in the form of a catenary consisting of precast prestressed concrete units supported on steel ribs. The cladding would support itself, where

he wind stresses could be taken up by the steel arch ibs. Such a design, while giving some of the advantages of a monolithic shell, would also be simpler to onstruct, for it would not require heavy shuttering.

Mr. Hiley's Reply

Mr. HILEY, in his reply, first thanked the President nd the other gentlemen who had contributed to the liscussion, for their kindly references.

Dealing with Major Braine's questions, he apologised or having used the wrong initials when referring to the ate Mr. Watson. He was, of course, referring to the ather, Mr. J. D. Watson, a Past-President of the nstitution of Civil Engineers, whom he knew very vell, and not his son Mr. D. M. Watson, the Vice-President of that Institution, who was in partnership vith his father.

Complimenting Major Braine on the work at Tel Aviv, ie said that anyone who harnessed power from any available source was doing a good engineer's job. To save—as Major Braine had intimated could be saved in London alone—gas from sludge equivalent to 55,000 tons of coal per annum, was a grand contribution and again emphasised what the author had tried to stress—that such by-products should not be neglected. A measure of the usefulness of what Major Braine had suggested was equivalent to 16 days running of a 360,000 kW generating station.

There were many matters, of course, which could not be covered in detail in his paper, but in finding some method of approach to the generation of electricity, he had mentioned various methods of power production of which he considered sludge gas as one. No doubt the subject of power production would not be exhausted if a dozen or so papers were written on the subject.

Mr. Bowen's point about the crane girder expansion joint was appreciated and promised to reproduce a detail of the method employed at Uskmouth.

Although a relatively small matter, it was a very important one since the smooth running of the engine-room gantry crane was dependent on a trouble-free joint. Stations with which Mr. Hiley had been professionally connected varied in design and he appreciated the reference to Hams Hall "B," which was an entire R.C. frame clothed in brickwork.

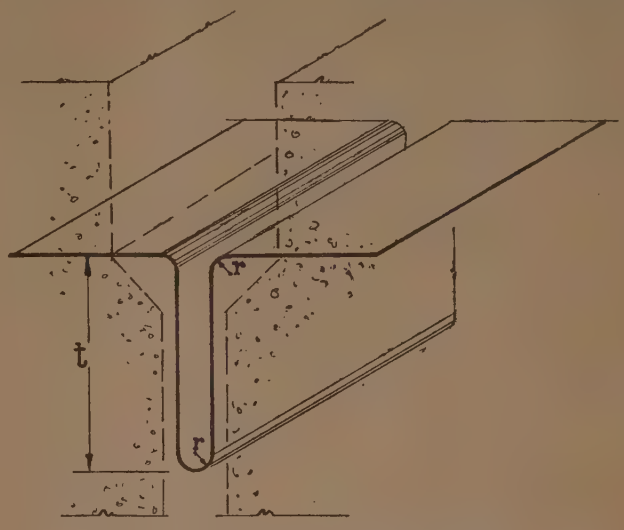
Mr. Bowen's reference to concrete casing to the engine-room columns was more a matter of cleanliness than one of providing the bare structural requirements. It would have been noted that the casing was carried up from floor level to the underside of the crane rail only. These compound steel stanchions, unless clothed as indicated, would have required painting and maintaining, so that taking the long view, the cumulative cost of difficult painting operations might well have been more than the initial clothing in concrete, to say nothing of the improved appearance.

Replying to Mr. Kent's remarks concerning the expansion joints, he said that two such joints were made dividing the main buildings into three sections. The joints ran transversely across the building and consisted of a 2 in. gap which housed a tongued copper joint, the ends of which were fixed at either side. The copper was bent as shown in sketch.

The radius of "r" was the same and the tongue "t" kept long so as to withstand the constant movement without fatiguing the copper. The actual movement was something under 1 in., but to house the joint, 2 in. was considered the minimum practicable.

As to the designed strength of the turbine house roof girders, only one of these—that over the loading bay was designed to take the load to the gantry crane which had to be lifted into position—the remainder were designed for roof loads only.

Regarding the boiler house roof girders at Uskmouth, where unit boilers were not employed, it was considered



Typical Section—Copper Strip at Expansion Joint
Fig. 2

an economy to provide some assistance in handling certain parts of the boiler plant. In schemes where unit boilers are used and where the boilers are hung from the building frame, such assistance is provided and forms a real economy in steelwork.

Replying to Mr. Joshi, Mr. Hiley writes :—

As summarised by Mr. Avi Joshi, it was usual to design a brick chimney so that under the worst wind conditions the entire section of chimney was under compression. A brick chimney was a gravity structure, stabilised by its own weight and was, therefore, *a priori*, an expensive form of construction. The cost of a brick chimney could be slightly reduced by allowing the neutral axis to fall within the section and neglecting that part of the section situated in the tension zone.

Mr. Avi Joshi had mentioned prestressed concrete as a possible material for use in chimneys. He (Mr. Hiley) was investigating this problem for future application. When using prestressing methods the compressive stress on the concrete would have to be much higher than those usually adopted in reinforced concrete and one was naturally led to examine the possibility of building a chimney by means of precast units.

The slide making comparisons of economy in chimney design was not included in the paper as printed and it was thought desirable to include it in the discussion.

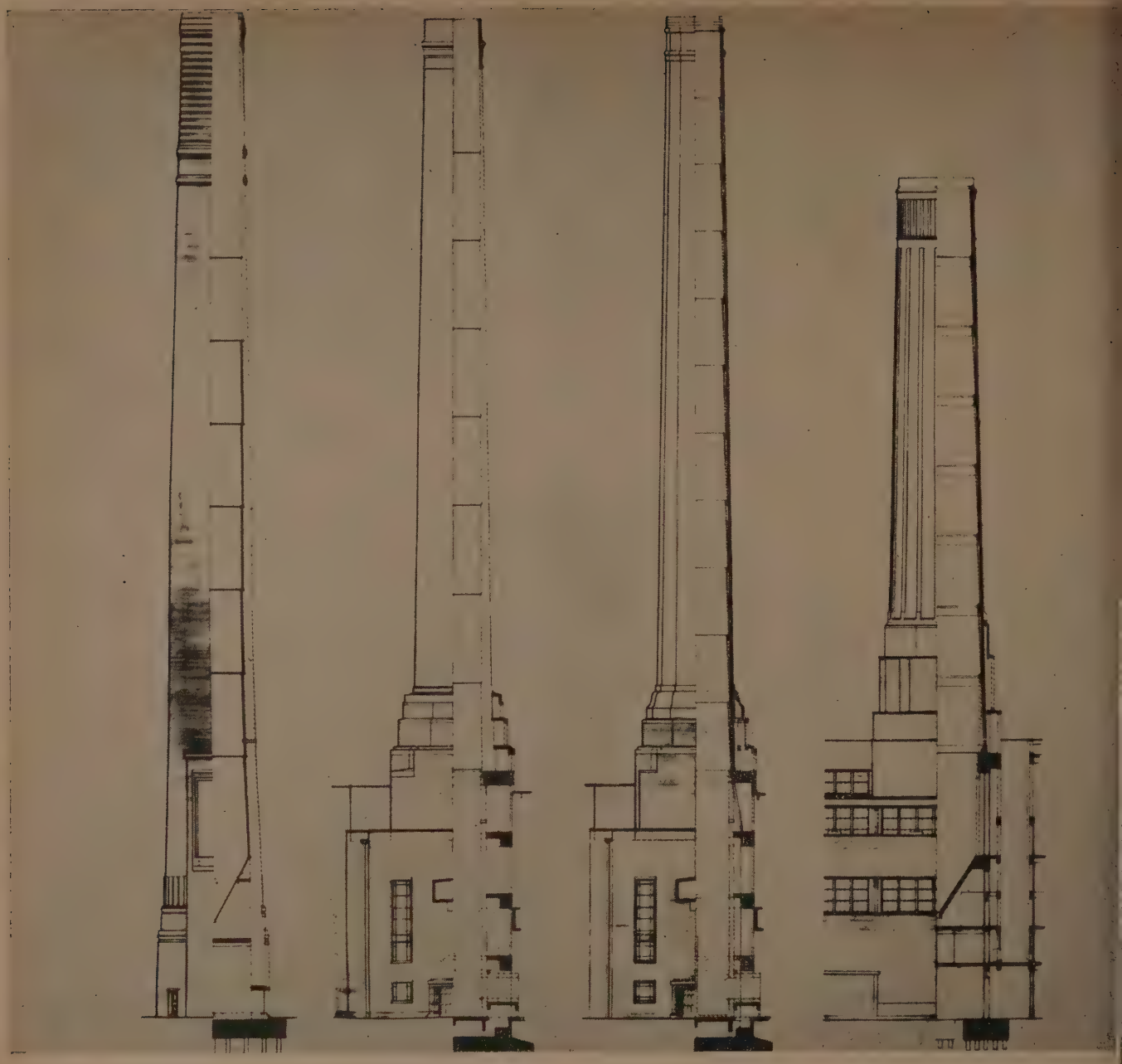
The Uskmouth chimneys were not prestressed but ordinary *in-situ* reinforced concrete lined as illustrated.

Regarding the point of economy in roof trusses, Mr. Hiley referred to his reply to a previous speaker. In the engine-room only one truss was strengthened to assist the raising of the gantry crane and that in the loading bay.

In respect of Mr. Joshi's last comment regarding conventional and advanced design of the building structure—he no doubt had in mind a suggestion put forward at the Joint Engineering Conference in 1951, where a parabolic or inverted catenary scheme was put forward for discussion.

Such departures from the traditional methods had to be very carefully examined indeed.

Where, in structures now being designed the boilers are suspended, the boiler house building columns on the



CHADDERTON POWER STATION	
240,000 KVA	
BRICKWORK CHIMNEYS	
Height	365 ft.
Dia. at top ..	19.25 ft.
Weight-shaft ..	7,000 tons

DRAKELOW POWER STATION	
240,000 KVA	
BRICKWORK CHIMNEYS	
Height ..	360 ft.
Dia. at top	18.5 ft.
Weight-shaft	3,400 tons
Stool, etc.	3,300 tons
TOTAL	6,700 tons

DRAKELOW POWER STATION	
240,000 KVA	
FERRO-CONCRETE CHIMNEYS	
Height ..	360 ft.
Dia. at top	18.5 ft.
Weight-shaft	1,950 tons
Stool, etc.	3,000 tons
TOTAL	4,950 tons

USKMOUTH POWER STATION	
360,000 KVA	
FERRO-CONCRETE CHIMNEYS	
Height ..	297 ft.
Dia. at top	25 ft.
Weight-shaft	1,900 tons
Stool, etc.	2,900 tons
TOTAL	4,800 tons

boiler frame are combined and this should make for economy.

Regarding the other large building unit that makes the whole structure—the engine-room and pump house—here we have gantries, the larger of which lifts up to 180 tons. Remembering this, the catenary suggestion does not seem to fit in.

If, as is suggested, the catenary idea is intended to save time of construction, one must measure this carefully. Progress on one part of a structure fully planted is more important than completing the building structure and then having to wait for plant foundations. One is inclined to measure progress by buildings only whereas real progress is in so planning, designing and carrying out the work that plant can be installed progressively, the builder following a sequence of wor whereby at all stages of building the plant foundatio is left behind for occupation by the plant contractors.

Such a sequence is taking place at Drakelow Pow Station, where in 2½ years full occupation to the plan contractor has been given to proceed uninterrupted with his installation.

Where a change in design is possible, is in the structur above the crane rails in the engine house and small units of work.

The PRESIDENT, at the conclusion of the discussio expressed appreciation to Mr. Hiley for the discussio which his paper had provoked.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of tructural Engineers was held at 11, Upper Belgrave treet, London, S.W.1, on Thursday, April 23rd, 1953, t 5 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that he elections as tabulated below should be referred to when consulting the Year Book for evidence of membership?

STUDENTS

BUTLER, Alyson Edward, of Harrow, Middlesex.
CARNAN, Lewis Rippon, of Consett, Co. Durham.
COOKE, Brian Herbert, of London.
CWALINO, Miroslaw, of London.
DODD, James Michael, of Wirral, Cheshire.
D'SYLVA, Eunan Declan, of Bombay, India.
EGWUATU, Gordon Chukwuka, of London.
FEAKES, Gerald William Ernest, of Reading, Berks.
HALL, Edward Tuffnell, of Walsall, Staffs.
HAWKES, John Murray, of Loughton, Essex.
HINDLE, Anthony, of Preston, Lancs.
JOHNSON, Brian, of Gateshead, Co. Durham.
KAY, Terence, of Sheffield.
LEVICK, Sidney Isaac, of Cape Town, South Africa.
STAPLES, Graham George, of Nuneaton, Warwickshire.
THISTLETHWAITE, Gerard, of Ormskirk, Lancs.

GRADUATES

ALAM, Aftab, of London.
ALEXANDER, George, M.A.(Cantab.), A.M.I.C.E., of London.
BATES, Peter John, of London.
BRATT, Harry Nigel, B.Eng.(Civil) Liverpool, of Rising Bridge, Lancs.
BRISTOW, Ronald George, B.Sc.(Eng.) London, of Bexley, Kent.
CAPELIN, Dennis Edwin, of London.
DEAN, Reginald, of Winsford, Cheshire.
GLOVER, Brian, B.Sc.(Eng.) London, of Landywood, Staffs.
HAVELOCK, John Henry William, B.Sc.(Eng.) London, of Bromley, Kent.
HEYWORTH, John Desmond, B.Sc.(Tech.) Manchester, of Rossendale, Lancs.
JOHNSON, Edward John, of Sidcup, Kent.
LONGMAN, Donald James, B.Sc.(Eng.) London, of Beckenham, Kent.
MORGAN, David Henry, of Newport, Mon.
PATEL, Indubhai Varadhbhai, B.E. Poona, of Derby.
SIMONS, Noel Edward, M.Sc. Natal, of London.
STRINGER, Robert George Alexander, A.M.I.Mun.E., of Sittingbourne, Kent.

MEMBERS

KERENSKY, Oleg Alexander, B.Sc.(Eng.) London, M.I.C.E., of London.
MACKINTOSH, Coleridge Stewart, B.Sc.(Civil) Rand, A.M.I.C.E., of Pretoria, South Africa.

TRANSFERS

Students to Graduates

BECK, Roy Cecil, of Norwich, Norfolk.
JULIER, Joseph Martin, of London.
PUN YIN KEUNG, D.I.C., of London.
TRUMP, Vernon Henry, of Ilford, Essex.
TYPROWICZ, Tadeusz Wladyslaw, of London.
WEAVER, Charles Louis Harry, of London.

Graduates to Associate Members

BRUNDRITT, Alan, of London.
GROOME, Leonard William Edward, of London.
LOACH, Douglas Clifford, of Littleover, Derby.
PEARCE, David John, B.Sc.(Tech.) Manchester, of Manchester.
SPEED, Cecil Harry, of Derby.
TWELVETREES, Walter Sydney, of Murton, near Swansea.

RE-ADMISSION

Associate-Member

DAVIES, Louis Edward, of Southport, Lancs.

OBITUARY

The Council regret to announce the death of JAMES FREDERICK WARD RUSSELL (Member), ARTHUR REES FAIRBAIRN, GEORGE DOUGLAS HAMILTON, STUART KILSBY MOBBS (Retired Members), and LEONARD WILLIAM ELLIOTT (Associate-Member).

RESIGNATIONS

Notification was given that the Council had accepted with regret the resignations of COLERIDGE STEWART MACKINTOSH (Associate).

JUNE MEETING

An Ordinary General Meeting of the Institution, for the election of members only, will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, June 25th, 1953, at 5 p.m.

EXAMINATIONS—JANUARY, 1953

The Examinations were held in January, 1953, at the usual centres in Great Britain, and at the following overseas centres: Beirut, Bloemfontein, Bombay, Bulawayo, Calcutta, Cape Town, Christchurch (N.Z.), Colombo, Delhi, Durban, Hong Kong, Jerusalem, Johannesburg, Kaduna, Karachi, Kelantan, Kuala Lumpur, Lahore, Lucknow, Madras, Miri (Sarawak), Montreal, Mount Isa (Australia), Nairobi, Salisbury (S. Rhodesia), Singapore, Tamale, Tel Aviv, Toronto, Wad Medani, Wellington (N.Z.).

Seventy-eight candidates took the Graduateship Examination, 60 at Home Centres and 18 overseas.

Three hundred and twenty-seven candidates took the Associate-Membership Examination, 248 at Home Centres and 79 overseas.

The total number of entries for both examinations was thus 405. Of these, 60 passed the Graduateship Examination, and 111 passed the Associate-Membership Examination.

The names of the successful candidates are :—

GRADUATESHIP EXAMINATION

ABRAHAMS, Frank Dudley.
AGER, John Stanley.
ALLANSON, Brian Melvyn.
ASTILL, Alan Walter.
BAKALA, Eugeniusz.
BEESON, Alan Spencer.
BENG WEE GIAP.
BEZUIDENHOUT, Willem Jacobus.
BOTING, Henk.
BROWN, Geoffrey Hewitt.
BUXTON, Leslie Gordon.
CAWOOD, Peter Ireland.
CHAN WENG CHIU.
CLARK, Peter Jerome.
CORNISH, Richard Leslie.
COXHEAD, Peter Leonard.
CRAGG, Wilfred Shipham.
DABROWSKI, Zbigniew.
DAINES, Alfred Lionel.
DICKSON, James.
DOORLY, Dermot Kieran.

GRADUATESHIP EXAMINATION—*continued*

ELDRIDGE, Frederick William.
FERSZTAND, Jakob.
FOLLY, Eric John.
GARRITY, Albert Francis.
GOODIER, Roger Ewart.
GOYAL, Ram Sarup.
HOMER, Albert Leslie.
JOSHIRAO, Chintaman Moreshwar.
KAPKA, Jerzy.
KOH BENG SWEE.
KOMORNICKI, Wladyslaw Stanislaw.
KWIECINSKI, Franciszek.
KYCIA, Serafin.
LANGHAM, Bernard Trevor
LEROI, Jules Anne.
MACGREGOR, David.
MCKEVITT, Vincent James
MORGAN, Albert Edward.
NARAYANA, K. Yogna.
OLIVER, Carl Richard.
PURVER, Gordon Douglas.
RADCLIFFE, George Frederick.
RAMSDEN, Jack Kenneth.
RAWNSLEY, Peter Michael.
READER, David.
REFFITT, Peter.
SAGEER, Mohd Mian.
SHERLOCK, William Albert.
SMITH, Ronald Walter.
STANLEY, Brian William.
STONE, Geoffrey Jones.
SUTCLIFFE, Roy Holtham.
TAYLOR, Phillip Austin.
TONGE, Brian Yardley.
TREICHEL, Jan.
TREZISE, Bernard.
VIELROSE, Witold Leon Ryszard.
WILSON, Russell.
WRIGHT, Frank.

ASSOCIATE-MEMBERSHIP EXAMINATION

AAGAARD, William Valdemar.
ALTERMAN, Israel.
ARD, Donald Leonard.
ARMSTRONG, George Herbert.
ASZ, Leon.
ATKINS, George Frederick.
AUSDEN, Alistair Charles.
AUSTIN, Michael.
BAGULEY, Maurice Grant.
BAIRD, James.
BAKER, Alexander John.
BAKER, Gordon Arthur Henry.
BALLARD, Eric Hugh.
BANFIELD, Arthur Reading
BELCHER, Charles Duncan.
BIRCH, Norman.
BOXALL, Charles Alfred.
BRAY, William Albert.
BRILL, Herbert.
BROOKE, Jack.
BURGE, Stuart.
BURR, Robert William.
CAWOOD, Peter Ireland.
CHANDRANGSU, Sirilakn.
CHENHALL, Raymond Leslie.
CLARK, Tony Allen.
CLUGSTON, Douglas Walker
COLLINS, Peter Courtney.
COLQUHOUN, William Alexander Smart.
DANAHER, Martin Denis.
DAVIES, Brian Clifford.
DERRINGTON, John Anthony.
DESHMUKH, Madhukar Dattatray.
DHAVALIKAR, Dattatray Ganesh.
DIVERALL, William.
DUNN, Allan George.
DUTTON, Charles Norman.
EASTON, Edgar.
EVANS, George Harold.
FIELD, James.
GAMSE, Joel David Harold.
GLAZE, Albert Gordon.
GRADY, Jack.
GRAY, Robert.
GREEN, Edward Ellis.
GUDGEON, Donald Keith.
HAMER, Clifford.
HARVEY, Peter Lawrence.
HAYNES, Alan.
HEPPLEWHITE, Eric Anthony.
HIGGINSON, Leonard.
HODGKINSON, Arthur.
HORRIDGE, James Frederick.
ISAACS, Basil.
JAAP, John Hunter.
JACK, Alexander.
JENKINSON, Anthony Richard.
KANVINDE, Sitakant Shantaram.
KERR, Gordon Cecil.
KHARKAR, Meghashyam Shantaram.
LAMB, Allan Roy.
LEWIS, Lawrence Henry.
LEWKOWICZ, Janusz Jozef.
LINSDALL, Kenneth Charles.
LIVESEY, Keith Haydn.
MAUNDER, Augustine Hamilton.
MICHAU, Peter St. Cyr.
MODLIN, Sydney.
NANDI, Subir.
NEWTON, Christopher John Broughton.
NEWTON, David Wilson.
NICHOLSON, Thomas Hedley.
OATES, Ronald.
OLIVER, Carl Richard.
O'MEARA, Brian Patrick.
PARISH, Jack Leonard James.

ASSOCIATE-MEMBERSHIP EXAMINATION—*continued*

PAXTON, Ian Hamilton.
PECKHAM, Donald.
POGSON, Albert Leonard.
POMEROY, Albert Frederick.
POOL, James Fraser.
PRICE, Selwyn Lionel.
ROSE, Gordon Michael.
ROSENAK, Sidney.
ROTHWELL, Leslie Herbert
RUSSON, Robert George.
SCHOU, Jorgen Arthur Kjeld.
SHAIKH, Mohammad Abdul Wahab.
SHROTRI, Ganesh Shankar.
SINGH, Jattan.
SLOWIKOWSKI, Leszek.
SPOONER, Leslie Allan.
SRINIVASAN, Kilnagar Vedachala Iyer.
STEPHEN, Lawrence Ian.
STEWART, James Campbell
STOCK, David Athol.
STOCKS, Trevor.
STOTT, Harold Welch.
SUBBARAO, Tippur Narayanarao.
SZCZERBANIEWICZ-STEPHENS, Stanisla Jozef Michal.
SZPILEWICZ, Jan.
TAYLOR, James Edwa (Jnr.).
TEMPLE, Allan.
THORNTON, Paul Rossly
THRELFALL, Robert Malcolm.
TRANFIELD, William.
TSAO HSIEN-HWA.
WESTALL, John Butte worth.
WILLIAMS, Harry.
WILMAN, David Richar
WOOLNOUGH, Edwin Arthur.

PRIZE LIST—JANUARY 1953, EXAMINATION

The Council has awarded the following prizes connection with the Examinations held in Januar 1953 :—

ANDREWS PRIZE. (For the candidate who obtains th highest aggregate of marks in the Associate-Membersh Examination, passing in all subjects).

M. G. Baguley, of Bexleyheath.

HUSBAND PRIZE. (For the candidate who takes th whole of the Associate-Membership Examination, passe in all subjects, and obtains the highest marks in th paper "Structural Engineering Design and Drawing"

M. G. Baguley, of Bexleyheath.

WALLACE PREMIUM (SENIOR). (For the candidate wh takes the whole of the Associate-Membership Examin tion, passes in all subjects, and obtains the highe marks in the paper "Theory of Structures (Advanced)"

S. Nandi, of London.

WALLACE PREMIUM (JUNIOR). (For the most successfu candidate in the Graduateship Examination, passing all subjects).

F. D. Abrahams, of Edgbaston.

EXAMINATIONS—JULY 1953

The Examinations of the Institution will next be he at centres in the United Kingdom and overseas c July 14th and 15th (Graduateship), and on July 16t and 17th (Associate-Membership).

GOLD MEDAL OF THE INSTITUTION

The Institution Gold Medal has been awarded t Professor J. F. Baker, O.B.E., for his services t structural engineering science over a long period i connection with the Steel Structures Research Committe of the Department of Scientific and Industrial Researc and his continued interest and research into problems structural design as envisaged by the Plastic Theory.

CERTIFICATE OF COMMENDATION

A Certificate of Commendation has been awarded t Monsieur I. Leviant for a paper entitled "Introductio to the Vacuum Concrete Processes."

SILVER SESSIONAL MEDAL

The Silver Medal for the best paper read before th Institution during the Session 1951-52 has been awarde to Mr. F. R. Bullen for a paper entitled "Unusu Design for a large Constructional Shop."

WARD OF MERIT AND SCHOLARSHIP FUND

The Council of the Institution has initiated an Award Merit and Scholarship Fund which it is hoped will cumulate funds from which monetary awards may be ade to the winners of medals, scholarships or prizes, or r other services rendered to the Institution or the ofession of structural engineering. The initiation of e Fund has been made possible through the generosity Mr. L. Scott White, O.B.E., a Past-President of the stitution.

The first award, of Ten Guineas, has been made to r. A. Bolton, for a paper on " A New Approach to the lastic Analysis of Two Dimensional Rigid Frames."

SPECIAL COMMITTEE ON AWARDS

The Council has constituted a Special Committee on wards with the following Terms of Reference :—

" To give consideration annually to the question of ecial awards, namely :—

1. Institution Gold Medal. Available for award y the Council to a member who has rendered to the stitution or to Structural Engineering services of xceptional and outstanding merit.

2. Institution Certificate of Commendation. vailable for award by the Council to a non-member ho has rendered to the Institution or to Structural ngineering services of outstanding merit.

3. Grants from the Award of Merit and Scholarship Fund. Available for award annually to the inners of medals, scholarships or prizes, or for any ther reason.

The Special Committee is constituted as follows :—

he President (*ex-officio*)	Mr. E. Granter.
hairman of the Finance and General Purposes Committee	Lt.-Col. R. F. Galbraith.
hairman of the Membership Committee	Dr. S. B. Hamilton.
hairman of the Literature Committee	Mr. J. Singleton-Green.
hairman of the Science and Research Committee	Mr. W. E. Thorowgood.
hairman of the Education and Examinations Committee	Mr. L. E. Kent.
hairman of the Legislation Committee	Mr. J. Guthrie Brown.
hairman of the Practice Committee	Mr. L. Scott White.

REPRESENTATION

The Council has made the following nominations of nembers to represent the Institution :—

Civil Engineering Codes of Practice Joint Committee

Mr. Stanley Vaughan (Vice-President).

British Standards Institution Building Divisional Council

Mr. G. S. McDonald (Vice-President).

British Standards Institution Committee—Recommended Dimensions of Reinforced Concrete Structural Members

Dr. K. Hajnal-Konyi (Member).

Mr. Donovan H. Lee (Member of Council).

DRURY MEDAL AWARD

The fourth competition for the above award will take lace in 1953. The subject is the design of the structure f a new factory building. The material of construction s entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of tructural arrangement. Economy in the use of steel is n important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary ; envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution or, failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

MACLACHLAN LECTURE COMPETITION

The closing date for the receipt of entries for the MacLachlan Lecture Competition is Wednesday, March 31st, 1954. Particulars of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture, and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering so long as in every second year the subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s. 0d.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1954.

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1954.

2. The subject of the Lecture shall be on any aspect of structural engineering.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text; if they are essential they should be embodied in appendices. Photographs, drawings, graphs,

etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Wednesday, March 31st, 1954.

RESEARCH AWARDS

The Council have instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of construction ;

(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;

(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the science and art of structural engineering ;

(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1952, and September, 1953, is October 1st, 1953.

LONDON GRADUATES' AND STUDENTS' SECTION

A visit has been arranged to the works of Murex Welding Processes, Ltd., at Waltham Cross, Herts., on Saturday, July 18th, at 2.30 p.m. A coach will leave 11, Upper Belgrave Street, S.W.1, at 1.45 p.m., returning to London at approximately 6 p.m. The fare will be 6s. 6d., and as seats are limited, members of the Section who wish to attend are requested to write to the Hon. Secretary as soon as possible, enclosing this amount.

Hon. Secretary : J. F. S. Pryke, B.A.Hons., Bushcroft, Slipe Lane, Wormley, Herts.

CRICKET MATCH

The annual cricket match between an Institution of Structural Engineers XI and The Blue Circle Club will be played on Saturday, July 11th, 1953, starting at 11.30 a.m.

This will be the sixth game in the series, the Blue Circle Club having won four of the previous games and the Institution one. All the matches have been played at Snaresbrook, but it is probable that the 1953 match will take place on the Blue Circle Club's new ground at Bromley, Kent.

Cricketers in any grade of membership of the Institution who would like to play in this game should write to D. A. G. Reid, L.C.C. Brixton School of Building, Ferndale Road, S.W.4, who raises the Institution team. A wicket-keeper and one or two reliable batsmen are particularly needed.

The Blue Circle Club are most generous hosts and are prepared to entertain the players and their friends and other members of the Institution who wish to be present. Any visitors who wish to accept the Club's invitation should send their names to Mr. Reid not later than June 30th.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES BRANCH—GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES BRANCH

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111 Ext. 257.

Natal Section, Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

Bending Moments in Box-Shaped Reservoirs

By Professor Dr. Ing. Hermann Craemer

For the computation of stresses in box-shaped tanks, ethods of different kinds exist. The so-called rigid ethods take into account as many factors as possible, us arriving at fairly correct results, but they involve borious mathematics. On the other hand, the usual proximate methods give rise to simple formulas but, ving to the simplifications necessarily assumed in such ethods, they cannot claim to take full account of the astic behaviour of the structure. In the following ethod a middle course is adopted.

Since box-shaped reservoirs in reinforced concrete nerally form a square in their horizontal plan, we ill deal only with this case. Then, due to symmetry, e walls must deform without rotation of their vertical lges, i.e., they act as fully encastré ; the same holds course if the ground-plan forms a regular polygon.

If the reservoir is filled to the top, the intensity of ressure at a depth $h - x$ from its top is as indicated in e figure, i.e.,

$$(1)\quad p_x = p_0 \frac{h-x}{h}.$$

Ve now replace the slab by two systems of vertical and orizontal beams crossing each other and having the idths dx and dy, as it is well-known from Marcus' and ther theories of slabs ; we hereby dissolve the load into $_x$ and p_y for the vertical and horizontal elements spectively, i.e.,

$$(2)\quad p = p_x + p_y.$$

Moreover, we assume that the load p_y on a horizontal lement and therefore also the ratios p_y/p and p_y/p_x emain constant on its full length. In the direction x, owever, we assume the ratio of p_x/p_y to be variable, amely a function of x. The deflection of a horizontal lement in its centre line ($y = o$) thus is

$$(3)\quad w_y = \frac{p_y L^4}{384\, E I};$$

here I is the inertia moment of unit width.

The displacements, w_x, of the vertical elements obey he differential equation

$$(4)\quad \frac{d^4 w_x}{dx^4} = \frac{p_x}{E I}$$

low, strictly considered, the displacements of the ertical elements should be the same as those of the orizontal ones at any point of the slab. In order to atisfy this condition, however, the ratio p_x/p_y should e considered as variable also in the horizontal sense nd in this way we should come to the rigid solution e just wish to avoid for its laboriousness. On the ther hand our assumption of a ratio p_x/p_y dependent x allows us to fulfil the condition of equal deflections ong the whole centre line, $y = o$, i.e.,

$$(5)\quad w_x = w_y.$$

he method therefore is more accurate than the approxiations used till now, in which the equality is assumed only at a single point. We therefore insert eq. (2) into (3) and the result into (4) and obtain the common differential equation :

$$(6)\quad \frac{d^4 w}{dx^4} + \frac{384}{L^4} w = \frac{p}{E I}.$$

in which w_x and w_y have been replaced by w.

In order to solve it we use the following abbreviations

$$(7)\quad \xi = \frac{3{,}13x}{L},\quad \varepsilon = \frac{3{,}13h}{L} \text{ and}$$

$$(8)\quad C\,\xi = e^{-\xi}\cos\xi,\ S\xi = e^{-\xi}\sin\xi,\ (C+S)\,\xi = C\,\xi + S\xi.$$

These functions have the advantage that, with increasing argument, they very rapidly tend to zero, thus greatly simplifying the arithmetic.

Tables for these functions are available in many text-books and manuals. The solution is :

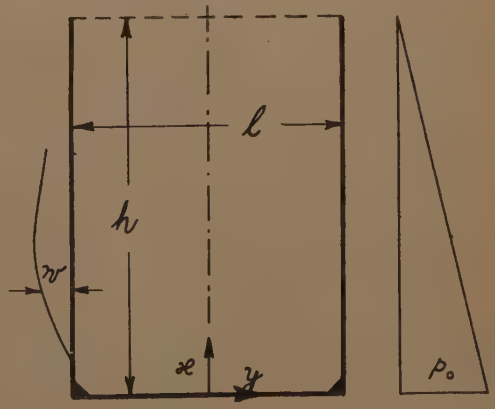
$$(9)\quad w = \frac{p_0 L^4}{384 E I}\left[\frac{h-x}{h} + K_1 C\,\xi + K_2 S\,\xi + K_3 C(\varepsilon-\xi) + K_4 S(\varepsilon-\xi).\right]$$

K_1 up to K_4 being constants. By computing the fourth derivation and inserting it into eq. (6) the correctness of the solution (9) can easily be proved.

The constants K are to be computed from the boundary conditions. Usually the bottom edge of the tank is considered as fully encastré. Thus, when $\xi = o$, $w = o$ and $dw/d\,\xi = o$ or, after carrying out the differential operations

$$(10a)\quad o = 1 + K_1 + K_3 C\,\varepsilon + K_4 S\,\varepsilon \text{ and}$$

$$(10b)\quad o = -\frac{1}{\varepsilon} - K_1 + K_2 + K_3 (C+S)\,\varepsilon - K_4 (C-S)\,\varepsilon.$$



If the reservoir is open at the top, $\xi = \varepsilon$, both bending moment and shear force or the 2nd and 3rd derivations must be zero ; therefore

$$(11a)\quad o = K_1 S\,\varepsilon - K_2 C\,\varepsilon + K_4 \text{ and}$$

$$(11b)\quad o = K_1 (C-S)\,\varepsilon + K_2 (C+S)\,\varepsilon + K_3 + K_4.$$

If, however, the box has a ceiling at the top we may assume the walls to be freely supported by it ; thus,

for $\xi = \varepsilon$, deflection and bending moment or w and its 2nd derivation must vanish, and we have

(12a) $o = K_1 C\varepsilon + K_2 S\varepsilon + K_3$ and

(12b) $o = -K_1 (C + S)\varepsilon + K_2 (C - S)\varepsilon + K_3 - K_4$.

If the dimensions of the reservoir are known the constants can easily be calculated from either the groups (10) and (11) or (10) and (12). Let us consider an open box with $h = L$ (a cube) ; thus $\varepsilon = 3{,}13$. Then $C\varepsilon =$ $-0{,}0437$, $S\varepsilon = +0{,}0006$, $\frac{1}{\varepsilon} = 0{,}32$. Then, from eqs. (10) and (11) we obtain :

$K_1 = -1 + 0{,}0437\, K_3 - 0{,}0006\, K_4$,
$K_2 = +0{,}320 + K_1 + 0{,}0431\, K_3 - 0{,}0443\, K_4$,
$K_4 = -0{,}0006\, K_1 - 0{,}0437\, K_2$ and
$K_3 = 0{,}0443\, K_1 + 0{,}0431\, K_2 - K_4$.

This system of simultaneous equations is best solved by iteration according to the following series

$K_1 =$	-1			$-1{,}003$	$-1{,}005$
$K_2 =$	0	$-0{,}68$		$-0{,}678$	$-0{,}690$
$K_3 =$	0		$-0{,}07$	$-0{,}104$	$-0{,}106$
$K_4 =$	0		$+0{,}03$	$+0{,}031$	

therefore

(13) $K_1 = -1{,}005$, $K_2 - 0{,}690$, $K_3 = -0{,}106$, $K_4 = +0{,}031$.

The greater ε is, the simpler the calculation of the constants will be. For tall boxes with a great ε the exponential functions can be taken as equal to zero and we get from eq. (10)

(14) $K_1 = -1$, $K_2 - 1 + \frac{1}{\varepsilon}$

while from both of the eqs. (11) and (12) we get

(15) $K_3 = K_4 = 0$.

Now the bending moments and shear forces m_x and q_x, consider for unit width are in the vertical elements

(16) $m_x = -E\,I \dfrac{d^2w}{dx^2}$ and

(17) $q_x = -E\,I \dfrac{d^3w}{dx^3}$.

Inserting eqs. (7) and (9) we get the general formul for the bending moment

(18) $m_x = -\dfrac{p_o L^2}{19{,}6}\Big[K_1 S\xi - K_2 C\xi + K_3 S(\varepsilon - \xi) - K_4 C(\varepsilon - \xi)$

The maximum negative moment occurs at the botton $\xi = 0$. For a tall reservoir, if esq. (14) and (15) a valid, we get from eq. (18) :

(19) $m_{xo} = -\dfrac{p_o L^2}{19{,}6}\left(1 - \frac{1}{\varepsilon}\right)$.

From the above example we thus get $m_{xo} =$ $-0{,}0345 p_o L^2$, the more correct value from eqs. (1 and (18) being $m_{xo} = -0{,}0355 p_o L^2$; the coincidenc is very close. The preceding equations show a clea analogy to the well-known theory of cylindrical shell

The moments m_y from the horizontal frame actio are obtained in the simplest way from eq. (3). Sinc at midspan we have $m_y = p_y L^2/24$, we get using eq. (

(20) $m_y = \dfrac{p_o L^2}{24}\Big[1 - \frac{\xi}{\varepsilon} + K_1 C\xi + K_2 S\xi + K C(\varepsilon - \xi) K_4 S(\varepsilon - \xi)\Big]$

In the corners we have moments of twice this value an of the opposite sign. Instead of this for a tall box w may write

(21) $m_y = \dfrac{p_o L^2}{24}\Big[1 - \frac{\xi}{\varepsilon} - C\xi - \left(1 - \frac{1}{\varepsilon}\right) S\xi$

By deriving with respect to ξ and equating to zero w find the section of the maximum of m_y ; we get

(22) $S\xi = \dfrac{1 - C\xi}{2\varepsilon - 1}$.

One solution of this equation is $\xi = 0$; this yields minimum, namely zero. The other solution is easies got by iteration ; it indicates the spot where m_y is maximum. In the case dealt with above $\varepsilon = 3{,}13$ an $S\xi = 0{,}190\,(1 - C\xi)$ and the following iteration serie is obtained

$C\xi = +0{,}5 - 0{,}06 - 0{,}006 - 0{,}015$
$S\xi = +0{,}1 + 0{,}2 + 0{,}19 + 0{,}192$,

therefore $\xi = 1{,}65$ and hence $x : h = 1{,}65 : 3.13 =$ $0{,}53$. The further particulars follow from eqs. (2 and (21).

Book Review

The Calculation of Concrete and Reinforced Concrete Pipes, by A. Guerrin and G. Daniel. (Eyrolles, Paris. 312 pp. 10 in. × 6½ in.)

This book is intended for Consulting Engineers and manufacturers of concrete and reinforced concrete pipes. It deals in detail with the behaviour of pipes under working conditions and their external stresses, one of the most intricate problems of soil mechanics.

Theory and practice are described at length, and the authors draw a synthesis of calculations leading to the precise and practical determination of the various stresses acting on the pipes in relation to the thickness and elasticity of the pipes, the overloads and other variables.

These results lead to an investigation of the moment the normal and the shear stresses acting on the wall the pipes. The authors have introduced an origin method of calculation which should considerab simplify the work of the civil engineer when stresses of complex nature are encountered.

This treatise ends with the grouping of special calcul tions applied to particular cases such as ringed pipe egg-shaped pipes, bed-plated and beam pipes.

The subject matter is extremely well and clear expressed and contains numerous examples. It shoul appeal to the designer concerned with the computatio of concrete and reinforced concrete pipes.

P. J. G.

Alteration and Repair of a Reinforced Concrete Warehouse

By Stephen Revesz, M.Sc.(Eng.), D.I.C. (Graduate)

Introduction

During the course of several air-raids on Birmingham 1940, the greater part of Messrs. Fletcher Hardware, td., warehouse was destroyed by fire. Many gutted uildings collapsed, but a four-storey reinforced concrete uilding remained standing, although it had been also vept by fire from end to end. Many longitudinal racks appeared in the concrete, and permanent de-rmation occurred in some floor panels. After super-cial repairs in 1942, the floors were tested and passed his business from the old warehouse (outlined heavily on the plan Fig. 1) with the minimum of interference. It was not intended to renew this building, but carry out remedial measures as necessary. The paper below describes some structural alterations to ensure the safety of the old four-storey reinforced concrete warehouse.

Before the fire, a basement existed on the site of the proposed Stage II and III structures, but this basement was filled with the rubble of the collapsed buildings. The basement was not required for the new schemes, and

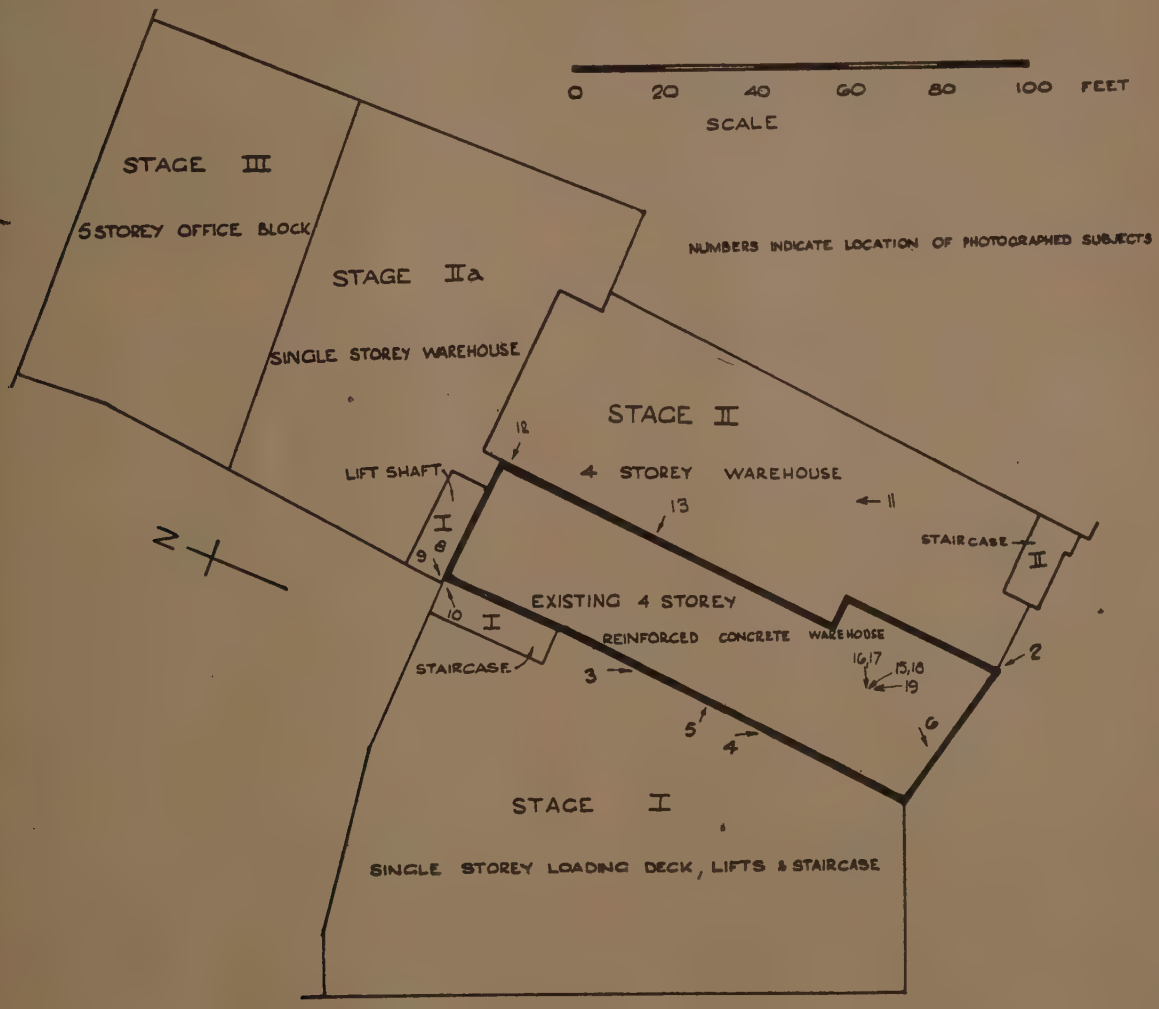


Fig. 1.—Sketch plan of general layout

s serviceable. By 1950, much of the repair work roved ineffective, and the corrosion of the reinforce-ment behind the spalling concrete required urgent ttention.

Reconstruction and extension of the original ware-ouse site was commenced at the end of 1950, in three onsecutive stages, in the order indicated on the sketch lan in Fig. 1. The building programme was limited y the client's primary concern for space to continue therefore *in situ* concrete piles were driven through the rubble from ground floor level.

Description of Old Concrete Warehouse

The four-storey reinforced concrete warehouse was constructed in 1919, on top of an existing basement and ground floor. A central row of columns, generally 12 in. × 12 in. section, divided the 28 ft.-wide concrete frames into two equal spans. The frames were spaced

at 13 ft. 6 in. and 20 ft. 6 in. centres, with secondary beams 12 in. deep and 5 in. wide between the frames, the floor thickness varying between 4 in. and 5 in. The

Fig. 2.—Loose rust surrounding column reinforcement

Fig. 3.—Galvanised steel mesh attached to exposed reinforcement

external columns on the west elevation were cut into the brickwork wall from first floor to basement level.

Where the concrete was broken out, it was found to contain ½ in. down granite chippings as aggregate : the concrete was very dense, apparently well compacted, and extremely hard. The reinforcement had been detailed in accordance with the Hennebique method, in which half of the longitudinal bottom reinforcement is bent up at the third points of the span, to form the top steel a the supports. Anchorage was provided by " fishtailing the bars. The shear reinforcement consisted of fla

Fig. 4.—Window beam broken up showing old reinforcement

Fig. 5.—Old concrete column after removal of surrounding brickwork

steel binders 1½ in. wide, ⅛ in. thick. The vertical ba of columns were butt jointed, and surrounded by loose fitting 6 in. long steel tubes, filled with grout.

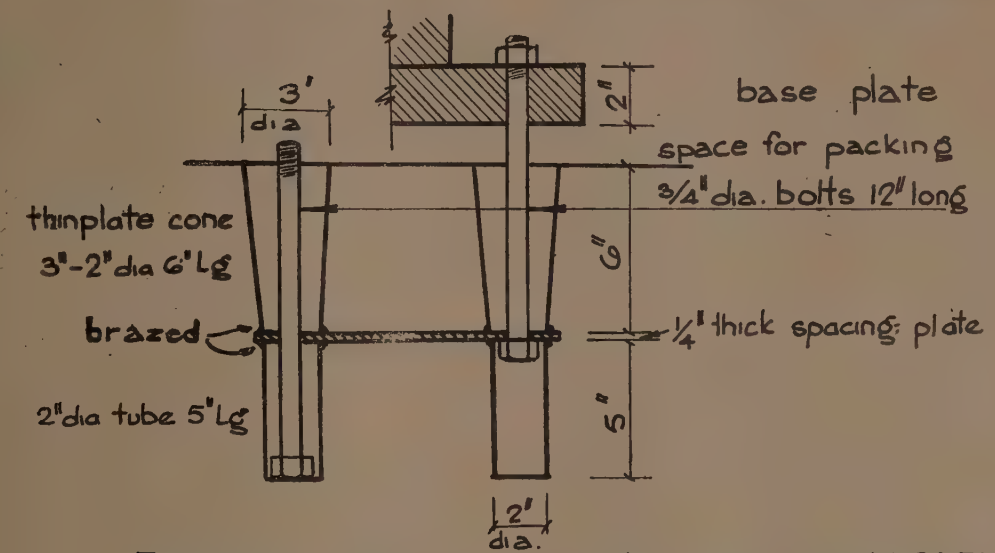
Reconstruction of Existing Beams and Columns

It was observed that the patchwork rendering appli in 1942 was becoming detached from the main body the concrete, due to the continuing corrosion of

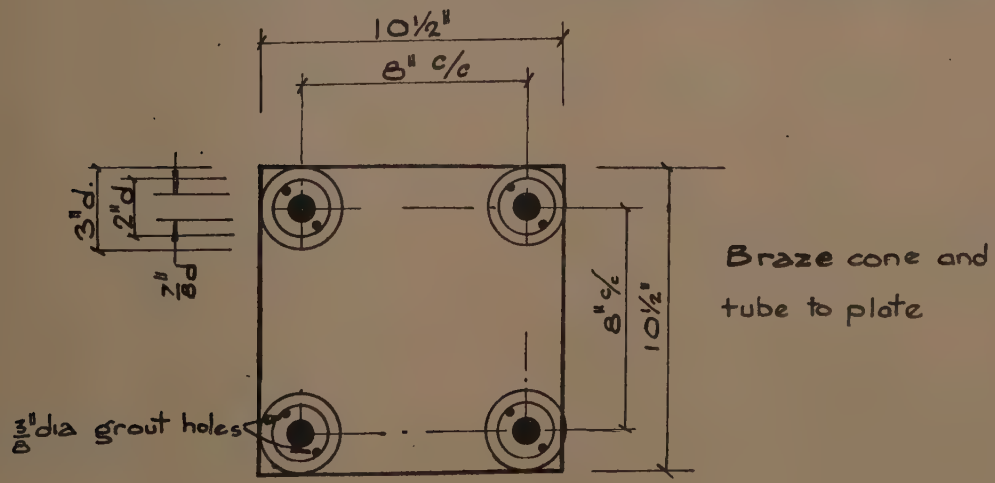
nforcement behind it. To prevent a recurrence of t trouble, three precautions were taken : (i) sufficient ncrete was cut out to ensure that the fresh material

Fig. 6.—Effect of corrosion on filter-joist

Fig. 8.—Steel stanchion being inserted into position



ELEVATION OF RETRACTABLE HOLDING DOWN BOLTS.



PLAN OF 1/4" t. MS. SPACING PLATE.

Fig. 7.—Sketch of retractable holding-down bolt unit

ipped the main reinforcement ; (ii) a heavy galvanised esh was tied to the main reinforcement, and thus the esh material was reinforced ; (iii) all oxidised steel was moved.

Fig. 2 shows the junction of the column and beams on the second floor of the warehouse, illustrating the condition of the column bars after removal of the loose rendering. Approximately half the sectional area of

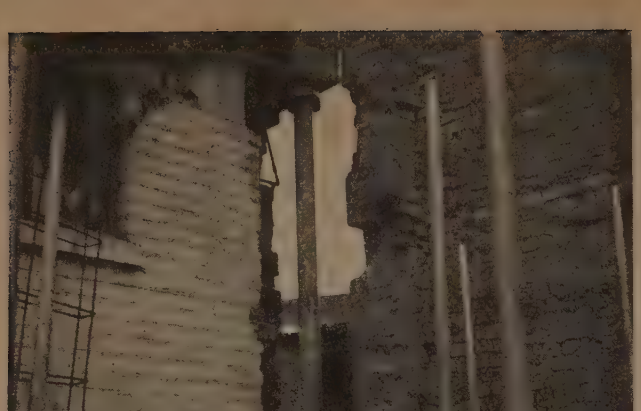


Fig. 9.—Stanchion in position after removal of trestles

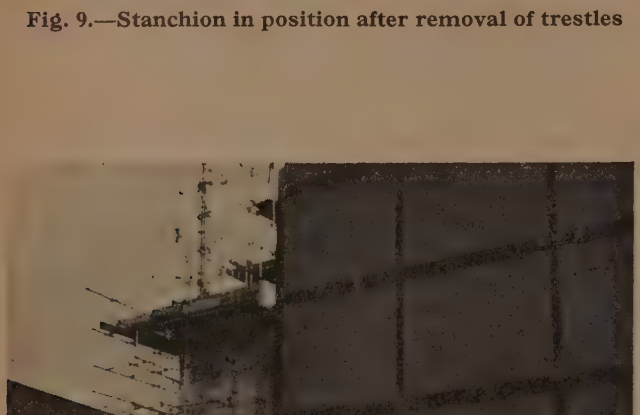


Fig. 10.—Stanchion encased. West elevation of warehouse

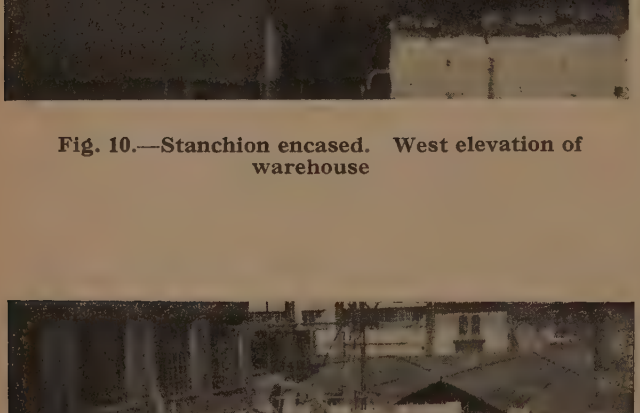


Fig. 11.—Cantilever construction of Stage II warehouse extension

the bar was lost due to corrosion. Experience on th site has shown that wire brushing in itself would n remove the rust entirely, which was contained in th "pores" of the metal. After trials, it became evide that a chemical containing a dilute acid in conjunctio

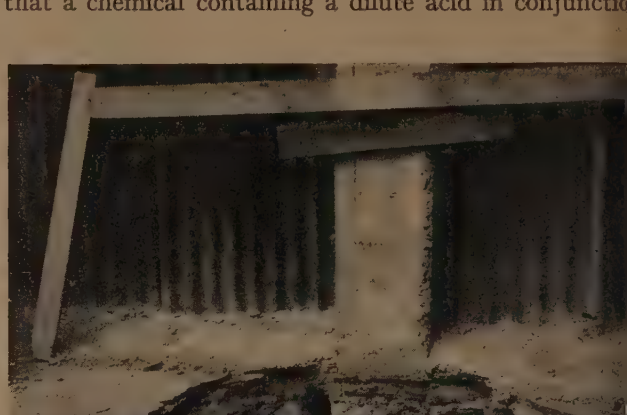


Fig. 12.—Paving slab and mass concrete under ground floor column

with an inhibitor, brushed on to the steel and wiped o would produce better results. Fig. 3 shows an extern beam, with galvanised mesh attached to the ma reinforcement, in the process of re-rendering the fac In some cases, the flat steel stirrups were reduced paper thickness by oxidisation, and cracking was severe that all the concrete was cut out (Fig. 4), ne steel added, and the beam re-concreted.

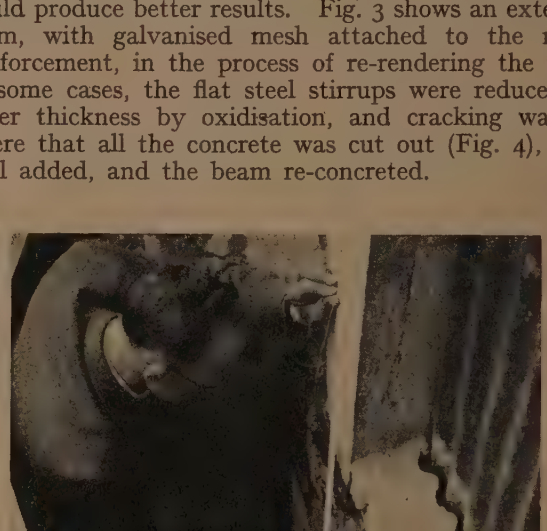


Fig. 13.—4 in. overhang of an old ground floor column

Fig. 5 shows one of the ground floor columns on t west of the building, which had been cast into the bric work in 1919. A steel mesh was wrapped around t column, and encased in 3 in. of concrete casing. C the ground floor, in the south of the warehouse,

ection of the existing filler-joist floor was demolished nd renewed in reinforced concrete. Fig. 6 shows a oist taken from the old floor, which joist clearly displays he effect of corrosion. The joists were at 5 ft. centres n the 5 in. floor, without reinforcement of any kind etween them. As a consequence, the floor panels ollapsed in whole during demolition ; caution had to be xercised by the demolition party, as large areas broke way without warning.

Underpinning of the North-West Corner

The 9 in. brick panel which covered the north and vest facade of the old concrete warehouse were scheduled

Fig. 15.—Old column at commencement of demolition October 10th, 1952

for demolition, to provide access into the building on the ground floor, and admit windows elsewhere. There was no column from the ground to the first floor in the north-west corner of the structure, the load being transmitted from floor to floor by the brickwork skin. It was intended to underpin the first floor corner of the structure by inserting a 5 in. square solid steel stanchion, 12 ft. 6 in. long. Trestles were erected in the lift shaft and staircase foundation raft north and west of the corner respectively. and two 15 in. × 4 in. R.S. channel sections used as needles to support the corner temporarily. The staircase wall was enlarged to form a concrete pier, on which the steel stanchion was to rest. A specially designed retractable holding down bolt unit (see Fig. 7) was cast into the top of the concrete pier. The purpose of the retractable bolts was to allow 1½ in. play only between the length of the stanchion and the space left for its insertion, so as to minimise the depth of packing (with steel wedges) to 1½ in. The recess into which the bolts slid was filled with liquid grout before the packing commenced. Fig. 8 illustrates the trestles which supported the corner from the north side, showing the manner of placing the stanchion in position, while Fig. 9 illustrates the stanchion after the removal of the trestles. The stanchion was later encased in concrete, and Fig. 10 shows the corner from the west, after the brickwork was removed. The work described was carried out by a sub-contractor at day-work rates ; the total cost was found to be appreciably below the lump sum estimate for the operation obtained from the main contractor.

Underpinning the Ground Floor Columns, East Side of the Old Warehouse

The new four-storey warehouse of Stage II was a cantilever reinforced concrete structure (see Fig. 11), founded on *in situ* piles. This system avoided interference with the existing foundations along the edge of the old building. The existing basement of the old building projected about 6 feet beyond the line of columns on the ground floor. The ground floor over this space was scheduled for demolition, to be covered by the cantilever system flooring of Stage II. While the old floor was being demolished, it was discovered that the old 11½ in. × 10½ in. reinforced concrete columns carrying the four-storey structure were not connected soundly with the basement piers. Apparently no starter bars had been used in the construction of the ground floor columns, because the paving slab continued under them, with mass concrete filling beneath, as shown in Fig. 12. There was also an eccentricity of the concrete construction, as shown on the photograph. In other cases, there was a dangerous overhang, the worst being 4 in. as shown in Fig. 13. The underside of the overhang was cut back at 45°, and a new concrete pier cast under it. The face of the old pier was hacked, holes drilled into the faces adjacent the side with the overhang, into which holes the reinforcement of the new pier were anchored.

Underpinning of the Double Column in the Old Basement

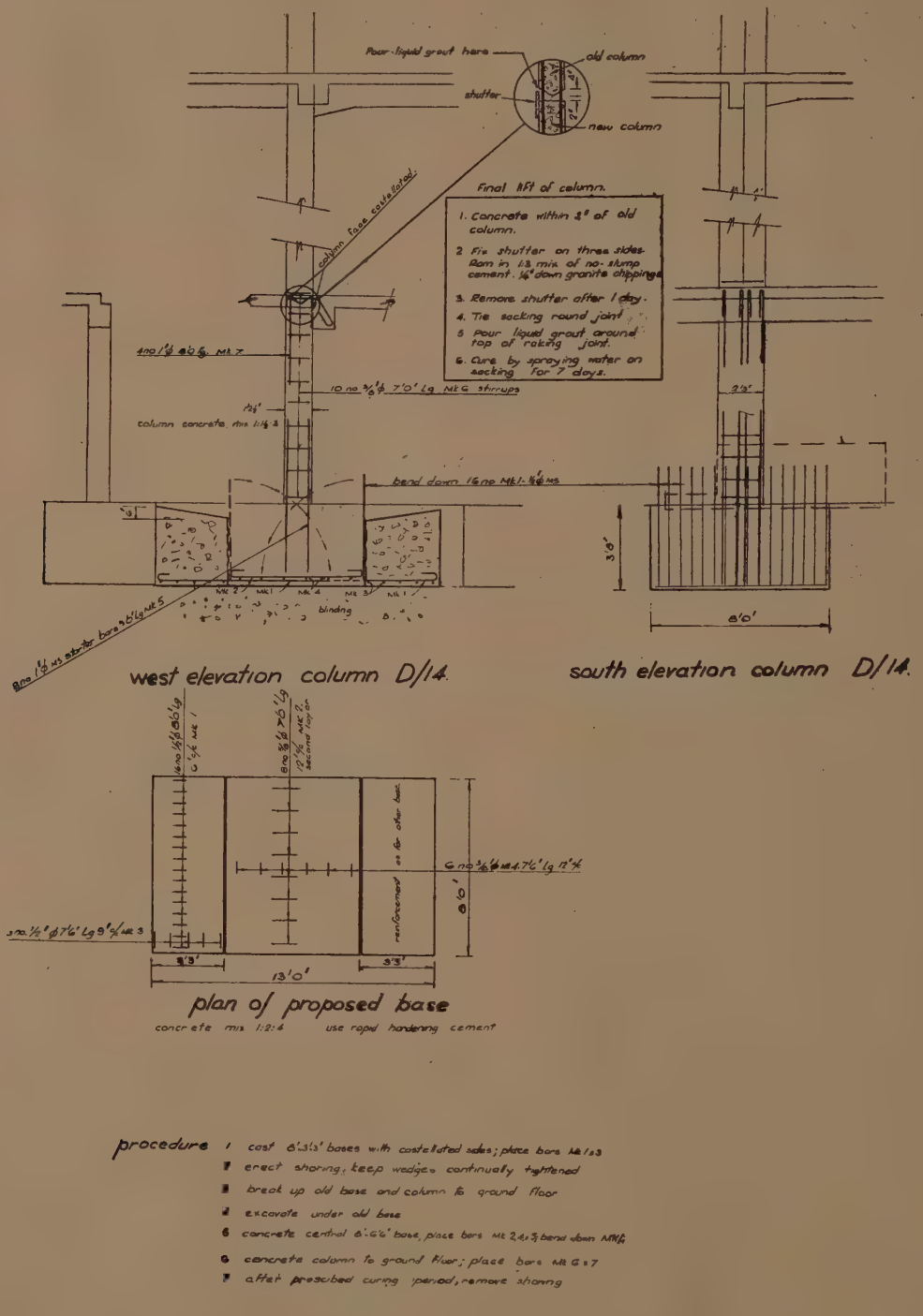
During the extension of the basement in the south of the old warehouse, it was found that the central column foundation rested on soft black earth. The base projected into the basement by about three feet high, and its underside was only a few inches below basement floor level. The size of the base was 10 ft. × 6 ft. It carried a double column, each about 1 ft. 2 in. × 1 ft. 1 in. which had been erected at different periods, and were separated by a narrow gap. It had been intended to place an additional footing onto the original foundation, and erect a 12 in. × 12 in. column adjacent to the existing double columns, to carry a proposed new ground floor inside the warehouse. In view of the bad soil beneath the base, it was decided to cut out both the original base and the double columns to ground floor, since three gains could be affected this way :—

(1) Access to the extended basement would be improved by removing the projection of the old base.

(2) By incorporating shear bars into the reconstructed column, the additional 12 in. × 12 in. column to the ground floor would not be required.

(3) Danger of further settlement could be eliminated.

The scheme for underpinning is illustrated in drawing Fig. 14, where the base is shown in relation to sections through the building. Firstly, the earth was excavated operation. Timber props were now erected from th basement to the top floor of the warehouse. This w necessary, because the basement props picked up th first floor beam soffits : the shearing force at the junctio of the beams and columns would have proved to excessive, and the load had to be transferred throug the beams. The wedges were tightened from the to floor downwards.


Pour liquid grout here
old column
shutter
new column
Final lift of column.
1. Concrete within 3" of old column.
2. Fix shutter on three sides. Ram in 1:3 mix of no. slump cement. ¼" down granite chippings
3. Remove shutter after 1 day.
4. Tie sacking round joint
5. Pour liquid grout around top of raking joint.
6. Cure by spraying water on sacking for 7 days.
cushion face castellated
4no 1½ dia bs Mk 7
10 no ⅜ sq 7'0" lg MK G stirrups
1½"
column concrete mix 1:1½:3
bend down 16 no Mk1-½ sq bs
3'3"
3'0"
8'0"
Mk 2
Mk 1
Mk 4
Mk 3
Mk 1
blinding
8no ½ sq bs after bars Mk 5
west elevation column D/14.
south elevation column D/14.
14no ½ sq 6'6" lg
6 ½ sq bs 1
8no ½ sq 7'6" lg
12 ½ sq MK 2 second top bs
6 no ½ sq bs 4 7'6" lg 12 ½
reinforcement as for other base
8'0"
3no 1½ sq 7'6" lg 9 ½ sq bs 3
5'3"
3'3"
13'0"
plan of proposed base
concrete mix 1:2:4 use rapid hardening cement

procedure

1. cast 6'3'3' bases with castellated sides; place bars Mk 1&3
2. erect shoring, keep wedges continually tightened
3. break up old base and column to ground floor
4. excavate under old base
5. concrete central 6'6" base, place bars Mk 2,4 & 3 bend down Mk 1
6. concrete column to ground floor; place bars Mk 6 & 7
7. after prescribed curing period, remove shoring

Fig. 14.—Scheme for underpinning column D14

on the north and south side of the old footing, to a depth of 4 ft. 6 in., where yellow sand was found. Two concrete bases were cast into these holes, with bent up reinforcement, and a castellated shutter board as shown. The tops of these subsidiary bases were finished at a rake, so as to pick up the timber props used in the

The concrete of the column was cut at ground floor level (see Fig. 15), leaving a blunt V-shape pointing downwards. When about a foot of concrete was removed by hand chiselling, the reinforcement was firmly tied together with timber packing and pipe clips, so as to reduce vibrations being transmitted, and

alling occurring at the corners of the column. This n was successfully achieved, despite the compressed pneumatic hammer used for breaking the concrete ee Fig. 16). An indication of the toughness of the old ncrete is illustrated by the fact that about 15 cutting ols were ruined in the operation. The earth between e subsidiary bases was excavated, the reinforcement nt down (see Fig. 17) starter bars added, and the

¼ in. down granite chippings, rammed into place using back and side shutters.

The semi-dry mixture was obtained in the following way : A bucket was filled with water to a given level, to which cement was added slowly, stirring all the time. When the grout had congealed into a consistency resembling dough, it was emptied on to a board, where the chippings were worked into it by hand.

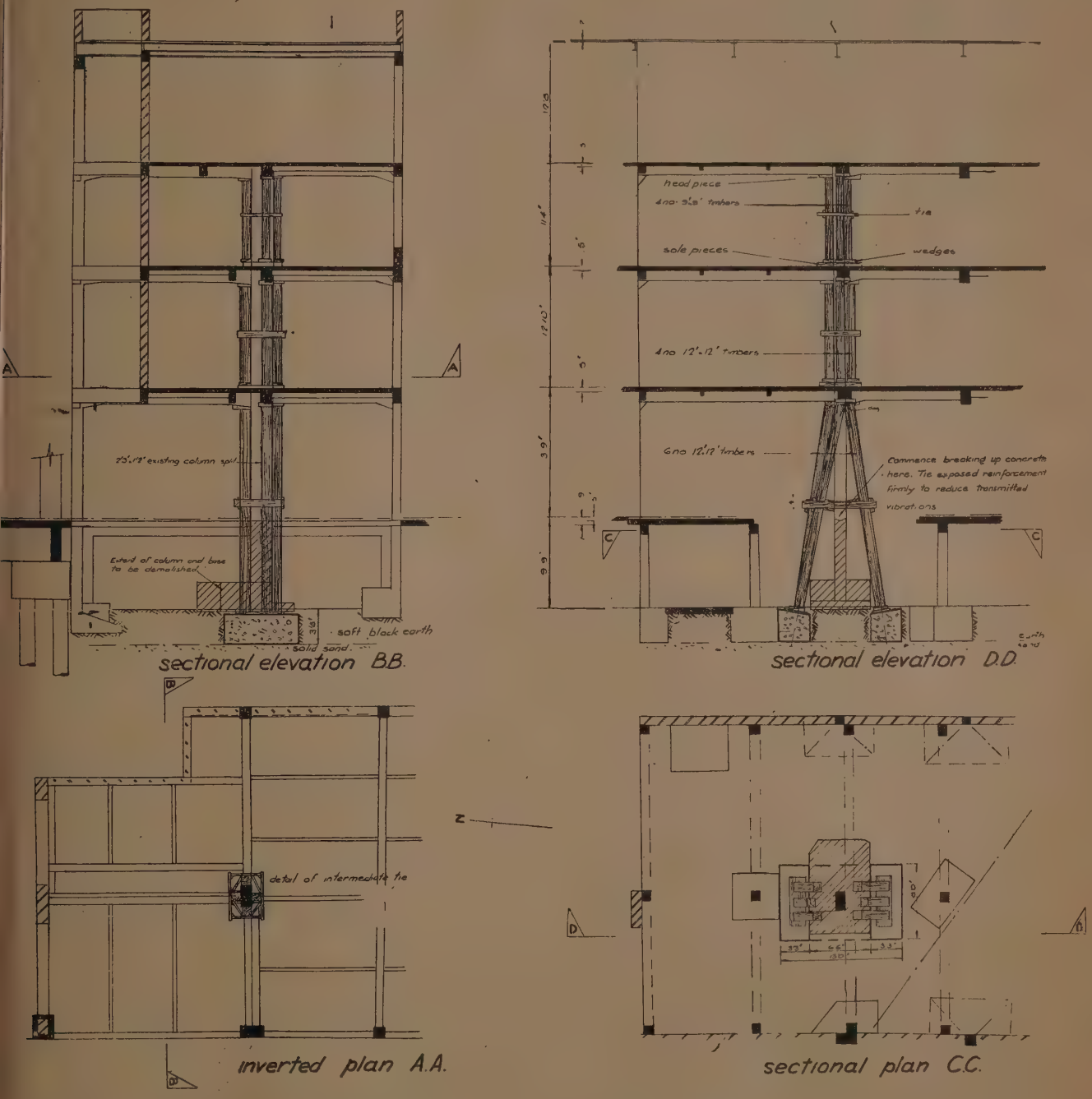


Fig. 14.—Scheme for underpinning column D14

termediate base concreted. Stirrups were placed round the reconstructed column bars, so as to unite the revious double column, as shown in Fig. 18. The last t of the column was cast to within 2 in.-3 in. of the stump of the old column. The remaining space was lled with a semi-dry 1 : 3 mixture of cement and

One day after ramming in the mixture, the shuttering was removed. Sacking was tied around the column immediately below the top of the V joint, where the granite mixture was not brought to a sharp point. Liquid grout was poured around the circumference of the column, retained by a groove formed by the sacking.

The sacking was kept moist for seven days. The props were removed after a further seven days (see Fig. 19). The column was checked against a marked level, from which it appeared that no settlement occurred in the operation, from beginning to end. The column had its

Fig. 16.—Demolition of old foundation October 21st, 1952

Fig. 17.—Bent down reinforcement in new footing October 25th, 1952

severest test immediately after the removal of the props, when it had an unsupported height of 23 feet from basement to the first floor. The ground floor was then constructed, on completion of the tricky underpinning operation.

Acknowledgement

Acknowledgement is made to the clients, Messr Fletcher Hardware Ltd., Birmingham ; to the Architec E. Goldfinger, Esq., D.P.L.G., L.R.I.B.A. (for who

Fig. 18.—First lift of reconstructed column October 30th, 1952

Fig. 19.—Reconstructed column with shear bars, November 15th, 1952

the author supervised the work), and to the Consulti Engineer, Dr. K. Hajnal-Konyi, M.I.Struct.I A.M.I.C.E., for permission to publish facts, drawings a photographs referred to in the paper.

The Strength of Brickwork

Discussion on Dr. F. G. Thomas's Paper*

The PRESIDENT proposed a vote of thanks to Dr. Thomas for having discussed and illustrated his interesting and very valuable work, and declared the meeting open for discussion.

Mr. F. W. DAVEY (Associate-Member), commenting that occasionally engineers had to make some estimate of the strength of brickwork in more or less pure bending, said that a short time ago he was asked by a firm making precast concrete products to compare a precast concrete manhole with a conventional brick manhole. Very little information on the subject was available, and tests were made in the laboratory of the firm concerned, the results of which he illustrated. Two manholes were built in 9 in. brick, their dimensions being 36 in. by 30 in., one of them in Fletton brick and the other in a good engineering brick from Bexhill. A 5 : 2 cement mortar was used and the manholes were tested at the age of about four weeks.

The first illustration showed the Fletton brick manhole. The test machine used was the ordinary one used for the B.S.S. standard tests on concrete pipes ; it applied a knife edge load on the centre, top and bottom. The failure of that manhole was then illustrated. Mr. Davey pointed out that failure had taken place in a fairly straight line through both the mortar jointing or the brick ; thus the mortar seemed to be roughly of the same strength as the brick.

The third illustration showed the result of the test on the stronger engineering brick, where the fracture invariably ran through the joints ; there was no fracture through any of the bricks.

From the results of the experiments he had tried to calculate the modulus of rupture. In the case of the Fletton bricks the modulus turned out to be 210 lb. per sq. in., for the engineering brick it was 356 lb. per sq. in., and for the precast concrete it was 940 lb. per sq. in. On testing a single Fletton brick, the modulus was shown to be 420 tons per sq. in.

Referring to an illustration in the paper of panel walls tested by applying a uniform load over the whole surface, Mr. Davey said that from the results he had tried to arrive at the modulus of rupture, and his figures were, for the 4½ in. wall, about 600, for the 9 in. wall, 335, and for the 13¼ in. wall, 500. Those figures were all considerably higher than those applying to his manholes, and probably the reason was that when the panels were tested the first crack appeared long before the final failure, whereas in the case of the manholes, due to the arrangement of the tests the final failure had occurred almost immediately after the first crack had appeared. So that possibly the lower figures were more important to engineers, as a guide to the actual strength of the brickwork.

The cracking of the Flettons, the weaker type of bricks, in more or less straight lines through brick and jointing indiscriminately was observed during the blitz. In houses which were blitzed and which were built in Fletton bricks and cement mortar, the cracking was of the same type, there being a few broad cracks through both brick and jointing, whereas in the case of the older buildings, in which lime mortar was used, almost invariably the joints had failed and the walls had disintegrated into separate bricks.

That fact led Mr. Davey to ask for Dr. Thomas's opinion as to whether bonding was really necessary nowadays. It was quite natural that it had arisen when our fathers were using weaker mortar, i.e., lime mortar, but was it really necessary to-day ?

Mr. A. L. HANDLEY, O.B.E. (Member), referred to the restriction, in the London Building (Constructional) By-laws 1952, that the minimum thickness of a calculated brick wall should not be less than one-sixtieth of its height. For example, this would limit the height of a 9 in. calculated wall to 45 ft. irrespective of the quality of the bricks and mortar used. He asked Dr. Thomas whether he considered this restriction necessary in the case of a calculated wall where the permissible stresses were not exceeded having regard to the limitations imposed by the slenderness ratios of the wall in its various storey heights.

Mr. E. G. ROBINSON (Associate-Member) said that most of the information one heard about load-bearing brickwork made no reference to chimney brickwork. The design considerations for such brickwork were rather different from those of ordinary load-bearing brickwork, in that one had to safeguard against it being too rigid, which meant that strong mortar could not be used. Therefore, the suggestion that comparatively high stresses could be imposed when using a weak mortar with strong bricks was of interest, in that it could make a very substantial difference to the design of a chimney. The present position was that chimneys, in the main, were designed first and calculated afterwards. But that took us up to only the first 200-300 ft. ; at greater heights the designer looked for guidance in the Code, but unfortunately he rarely found it.

The question of slenderness ratio in particular raised a knotty point. Most rules stated that the height of a chimney should not be more than 12 times its diameter at ground level ; it had nothing to do with the thickness of the wall. In some cases the height/thickness ratio might be as much as 140. A little while ago he had come across a design for a chimney 350 ft. high, having 2 ft. 6 in. thick walls at the bottom, giving a height/thickness ratio of 140, but the height/diameter ratio was 12.7. He had calculated the dead load on the bottom to be 142 lb. per sq. in., and making an assumption for wind pressure, the combined load was 262 lb. per sq. in. But none of that conveyed very much unless one knew what to do about the slenderness factor. Did the Code of Practice refer to the chimney dimension, or the wall dimension ? Assuming the former, the maximum combined equivalent unit of stress derived from the C.P. factor was 524 lb. per sq. in., which seemed to him to be rather high. In one handbook 350 lb. per sq. in. was noted as a reasonable working maximum.

Another matter of interest was the standard of workmanship, the difficulty of discovering what effect the tradesman had on the job he was doing. In chimney building, that question came very much to the fore, as a high degree of solidity was not desirable, and he felt that the use of a low load factor for the brickwork might with advantage be carefully considered. There was

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 12th, 1953. Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President) in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 2, pp. 35-46. (Feb., 1953.)

not the same risk of falling below a low standard as would be the case if a high standard were set. Application of a high load factor to low quality brickwork was probably desirable in general, bearing in mind that, the rougher the material, the rougher was the work, for the poorer bricklayer was employed on the lower quality work.

The Code of Practice, he continued, referred to concrete bricks as well as to clay bricks, but in Dr. Thomas's paper there was no reference to the testing of concrete bricks. It was fairly well known that some types of concrete brick were liable to crack in the work, which he was inclined to think was the result of lack of toughness in concrete as compared with clay. He was really concerned about the use of strong bricks in weak mortar, bearing in mind that the strength of a wall was probably related to the increased strength of the brick in shear ; he wondered whether any tests had been made to relate shear stress in different qualities of clay brick to final strength, particularly in connection with the use of weak mortar.

Finally, Mr. Robinson referred to the great difficulty which was bound to occur in connection with moisture. With cement mortar it was essential to wet an absorbent brick to prevent the water required for hydration of the cement being taken from the mortar by the brick. With lime mortar the reverse was the case, as one relied on the bricks to help dry out the mortar to give a reasonably early set. Many specifications required the wetting of all bricks, irrespective of their absorption or the type of mortar. If there were no absorption, the wetting of the brick could make it practically hopeless for the bricklayer to do anything with it at all.

A SPEAKER, dealing with the determination of the load factor, asked whether it was calculated on the basis of the load at which the first crack appeared or on the basis of the crushing load at final failure.

Another SPEAKER, discussing the assessment of eccentricity of loading on brick piers, asked for Dr. Thomas's comments in the case where one end of a concrete beam was supported across the top of a pier. With a flexible beam the interaction might be over half the width of the pier at the beginning of the loading ; but as the load was increased it might affect the pier sufficiently to reduce the eccentricity or to counteract its effect in large measure. He asked whether tests had been made in that connection.

The interaction of the various parts of a structure, he continued, was very important, and he recalled the author's illustration of the interaction of a brick panel in a concrete-clad steel frame. If we were to achieve greater economy in the design of multi-storey buildings, the interaction of the various parts had to be studied more closely and it seemed to him that that was the sort of work the Building Research Station could very well do ; probably it was being done there.

One aspect of the problem which he instanced was that of the strength of a steel joist embedded in a concrete floor. He suggested that the combined action of the concrete and the steel would give far greater strength than it was customary to allow at the present time.

Another aspect was that in a multi-storey building there were precast and hollow tile floors to carry partitions running in each direction over the floors. That also should be borne in mind in the consideration of the interaction of one part of the structure with another.

Mr. A. L. HANDLEY, O.B.E. (Member), asked if he were right in thinking that in the opinion of the Building Research Station a wall built of concrete blocks, which were comparatively large in size (say) 18 in. by 9 in. by 4 in., was stronger than the corresponding brickwork built with the normal 4½ in. brick. He raised the point because in these days cavity walls seemed to have a great many adherents, and the supervision of their construction was extremely difficult. He had in mind what he considered to be a bad practice, that of having a cavity wall built with a brick outer skin and with a block inner skin.

He had had under his supervision buildings in which that type of mixed wall construction was used. He did not know whether those responsible for adopting this form of construction thought that both skins would be built at the same time but in his experience this was not practical and he had never known it done.

To get a sound job, constant supervision was necessary, as unless carefully watched ties were apt to be left out at the time the outer skin was being built. Then it was frequently found that the block courses did not correspond with the brick courses and it was not unknown to find that the ties had been bent into the cavity. To ensure that the two skins were properly tied in such case was both costly and troublesome because of the additional cutting of the blocks which was necessary to take the ends of the ties. Again, the heaviness and unwieldiness of the blocks militated against their ease of laying and hence led to bad worksmanship.

Mr. J. S. SCOTT (Associate-Member) asked for Dr. Thomas's opinion about the London By-Law which provided that a load-bearing cavity wall must not be more than 24 ft. high and 30 ft. long.

Dr. THOMAS, replying to the discussion, first thanked all who had contributed to it.

Dealing with the requests for his opinion as to whether two particular limitations in the London Building (Constructional) By-Laws were necessary, he said, with regard to that mentioned by Mr. Handley, where the thickness of a wall was controlled in relation to its height, that in principle, of course, there was no reason to have any limitation whatsoever. If we knew the relationship between the slenderness of a wall and its strength, then the fact that a very slender wall had a very low strength could be allowed for in design, and so long as the permissible stresses corresponding to the high slenderness ratio were not exceeded, everything would be all right.

Data on the strength of slender walls, *as built in practice*, were however almost impossible to obtain ; and it was prudent therefore to introduce limits in practical design. The Code of Practice limits related only to the slenderness of walls between adequate lateral supports, but the L.C.C. By-Laws went a stage further with the limitation of slenderness irrespective of the spacing of such supports. It seemed to Dr. Thomas that the possibilities of accidental eccentricities of loading and of deviation of the wall from perpendicularity were likely to increase with overall wall height and that there was some justification for the additional limitation to which Mr. Handley referred. The limiting ratio specified was necessarily somewhat arbitrary and may well need to be modified in the light of experience.

With regard to Mr. Scott's question on the By-Law which provided that load-bearing cavity walls should not be more than 25 ft. high and 30 ft. long, he said that he could see no necessity for this limitation if the wall were calculated in accordance with the Code of Practice. He had been rather disappointed that the new L.C.C. By-Laws did not give the rules for designing cavity walls by calculation. However, the matter was linked with the limitation mentioned by Mr. Handley. For an 11 in. cavity wall, for example, the effective thickness given by the Code was two-thirds of 9 in.,

.e., 6 inches ; and hence with a limiting height-thickness atio of 60 the wall could not be built higher than 30 ft., vhatever the stresses were calculated to be. There was hus little to be gained by changing from the present ule-of-thumb method in the L.C.C. By-Laws to design f a cavity wall by calculation, unless thicker walls were ised. Nevertheless, Dr. Thomas would have preferred o have seen the Code rules for cavity walls adopted y the L.C.C.

With regard to the strength in pure bending, he had een most interested in Mr. Davey's illustrations. He ad assumed from Mr. Davey's remarks that the results iven were in terms of the maximum load. He quite greed that there was a considerable difference between he cracking load and the ultimate load for a panel uilt into a framework when subjected to lateral loading, o long as there was not a gap between the edges of the anel and the framework. If the edges of the wall were uttered to the framework, a slight bending of the anel would occur as the result of the loading, and then kind of dome action was set up whereby the panel ould carry very much larger loads than one would uppose from any simple calculation of plate action.

Dealing with the question as to whether bonding was ecessary, Dr. Thomas said he considered it to be essential. At the moment he could not see how one could conclude, from the tests which Mr. Davey had mentioned, that bonding was not necessary. The bending strength depended very much on how the loads were applied. In the tests on the manholes the line load was applied at right angles to the continuous horizontal joints ; the strength in bending of brickwork tested in that way must be much higher than when tested with the line load parallel with the continuous horizontal joints. He suggested that if a slab of brickwork were tested in the latter manner, instead of getting a result of (say) 300 lb. per sq. in., it would be more like 70 or 80 lb. per sq. in., or even less depending on the adhesion developed between the mortar and the brickwork. The adhesion was usually very poor, particularly where high strength brick was used with inadequate wetting, together with a strong mortar, without lime. It was necessary to be very careful, therefore, when using the figures for modulus of rupture which Mr. Davey had given. He did not want anyone to suggest that the next Code should allow high tensile stresses in brickwork.

In reply to Mr. Robinson, he said the Code of Practice CP111 did not really deal with chimneys at all. Nevertheless, he was very interested in Mr. Robinson's remarks, and said that in his opinion the slenderness ratio of a chimney should be based on the ratio height of the chimney to its overall diameter at the base, rather than on any consideration of the thickness of the walls. Really, it was a case where we should work on an L/k ratio rather than L/d. It was the radius of gyration which usually counted in determining the weaknesses of slender members, and the height/thickness ratio was used only for convenience in the design of some structural members. We avoided it in steelwork, but allowed it in concrete and brickwork, but he suggested that the best basis of design for chimneys would be a slenderness ratio where the height divided by the radius of gyration was used.

The question of shear strength and shear tests on brickwork was important. Very little work had been done in that direction in this country, but there was not the slightest doubt that in many cases the failure of brickwork resulted from shear. For that reason it was very important that an engineering brick should not be used with a straight 1 : 3 cement mortar, as had been the practice in some cases in the past. It was desirable to ensure that the bond strength between the brick and the mortar in both shear and tension was good, and that meant that when using engineering brick it was necessary to compensate for its rather low suction by introducing a certain amount of lime into the mortar.

Replying to the question concerning the load factors as shown in the diagrams, Dr. Thomas said those load factors had been worked out in all cases on the basis of the maximum load reached in the tests.

The difficulty of assessing eccentricity in practice, mentioned by another speaker, was a deterrent to the calculated design of brickwork. In recent years at the Building Research Station a fair amount of checking had been made of various structures, using load-bearing walls and piers, and they had their own kind of catalogue of assumed eccentricities. They would assume, for example, that the eccentricity on a wall carrying a concrete floor slab was less than that occurring when carrying a timber floor ; the greater flexibility of the timber joist, as compared with the concrete slab, would tend to give greater eccentricity. He agreed with Mr. Williamson that, as a wall bent, the eccentricity might tend to become less.

An interesting feature which became evident in some designs was that the eccentricity was not so important as might at first be thought, if it were assumed that eccentricity coming from a floor in a building disappeared by the time the floor below was reached. The Code allowed an increase of 25 per cent. of stress, when this was due to eccentricity of load ; and the effect of the eccentricity was often covered by this allowance.

Quite a lot of work was being done at the Building Research Station on composite action ; the field of investigation was wide, and he believed that the usefulness of it, if it could be codified, would be considerable, but on the other hand the difficulties of getting it codified were very great. This was however a matter for another meeting.

Dealing with Mr. Handley's comments on the combination of bricks and blocks in a cavity wall, Dr. Thomas said that he would not say that walls built of concrete blocks were stronger than those built with bricks. The Code of Practice did not differentiate between blocks and bricks in terms of the permissible stresses for particular strengths, except in one respect, and maybe the clause with regard to that had been misunderstood. It stated that if blocks were used where the height was at least twice the thickness we could use higher permissible stresses than those tabulated ; in fact, we could double them. It did not state that the blocks were stronger than bricks, but inferred that if a unit, whether of concrete or clay or other material, were tested as an individual unit and the height/thickness ratio was two, it was probable that the strength was only about half what it would be if we cut down the blocks so that the height/thickness ratio was similar to that of a brick.

He had been surprised by what Mr. Handley had said about the building of a cavity wall, and he must try to obtain more information about it. If he had been supervising the sort of jobs which Mr. Handley had mentioned, he would not have been very happy about them if the walls were built in the manner indicated.

Finally, Dr. Thomas thanked the meeting for its kind reception of the paper, and for the discussion, which would help him quite a lot and would give him several lines for further investigation over the next 25 years at the Building Research Station !

The PRESIDENT expressed the Institution's further indebtedness to Dr. Thomas for having answered the questions raised, and said he felt that all were satisfied by those answers.

Soil Mechanics in Relation to Structural Engineering

Discussion on Mr. P. L. Capper's Paper*

The PRESIDENT, expressing the thanks of the Institution to Mr. Capper for his review of the present position of soil mechanics in relation to structural engineering, said all would endorse his statement that close co-operation between practising engineers and research workers was necessary to fill in the gaps in our knowledge.

Supporting Mr. Capper's statement that too much care could not be taken in site investigation, the President said he felt that in some cases such investigation was not carried out with sufficient care, and in his view an engineering assistant should be on the site the whole time so that the fullest information could be obtained.

With regard to the selection of the positions in which borings were to be sunk, he said it was a difficult problem, and in that connection he was reminded that the late Sir William Matthews, referring to borings put down with the greatest care for the Outer Barrier, Hodbarrow, had said that if the engineers had purposely placed the borings so as to deceive the contractors with regard to the level of the clay bed, he did not think they could have placed them in better positions.

Mr. DONOVAN H. LEE (Member of Council) said Mr. Capper's most interesting paper was almost a text-book in itself. He need not have apologised for not running on into the practical applications; it had clearly been sufficient achievement to put forward so much within such a small compass.

In the Introduction Mr. Capper stated that : " It is now generally recognised that as regards the stability of foundations the theories of Rankine and Bell are not supported by experimental evidence." Mr. Lee said he might have read that sentence out of its context, but it struck him as being possibly a trifle misleading as most, if not all, practising engineers still used Rankine's formulas, especially for lateral pressures. Mr. Lee said he had looked in Mr. Capper's book to see what he said there and it was :

> " If due attention is paid to the limits of their applicability, Coulomb's, Rankine's and Bell's theories are still considered sound, and corrections are easily made when required to allow for deviations from the ideal assumptions."

Mr. Capper had spoken of the stresses in the soil beneath a footing being generally computed by Boussinesq's theory. Did he think it desirable to amplify that, because surely, it applied to the pressure at a point in an infinitely extensive elastic isotropic solid due to a point load and some compromise is needed for distributed loads.

Mr. Capper had said " For rigid footings on cohesive soil the pressure is greatest at the perimeter of the footing. . . " Mr. Lee asked if he would amplify tha because the examples in Fig. 1 showed the pressur variation for the unusual case of a soil loaded at th surface ; further into the soil the higher resistance fro the edges became relatively less and a fair distance i the ground whether the soil was cohesive or non-cohesive the pressures were tending to be uniform.

Another point which was of importance to practisin engineers was not what was the distribution of pressur (because most bases were designed for uniform pressur and very few failed) but whether on clay the loads wer approximately as calculated, after settlement with th passage of time. On clay a building was relatively rigid structure, and therefore the building tended t arch over the central area, or where settlement woul have been greatest with great shear and bending stresse sometimes coming into play ; the loads on particula bases might therefore alter considerably. On the othe hand, on granular soil the structure was flexible relativ to the soil.

Speaking of Housel's perimeter-shear theory, h welcomed and endorsed the author's reference to it an suggested it had not been sufficiently appreciated i this country.

Next, he asked if Mr. Capper believed, as he did, tha unconfined soil tests, generally speaking, gave pessi mistic results. It seemed that however careful w were in taking a sample it was never so good as it wa when in the ground.

Finally, he wondered whether Mr. Capper would lik to comment on M. Caquot, whose work in connectio with internal friction and earth pressures he had notice was not referred to although given prominence for som time in France.

Mr. JOHN KING (Associate-Member) added his con gratulations to Mr. Capper on having got so much int his excellent paper, and mentioned two points whic we should do well to stress, particularly as the pape seemed to be of the type which the younger men woul read and look to for guidance.

The first was the knowledge of geology which all youn engineers should have, and especially of the rathe complex geology of the more recent alluvial deposit because many of us had to build in the London regio and it was a very tiresome place in which to build Mr. Capper had drawn attention to it in the paper, bu it was worth underlining.

The second matter, which deserved a little fulle treatment than Mr. Capper had given it, was that c soil description and classification, on which Dr. Terzagh had laid considerable stress, because if a soil wer accurately described and classified, perhaps accordin to the Casagrande method, the knowledge of those wh had gone before us could be drawn upon, so that we migh have an indication of what to expect from various soil in various conditions. It was also useful for the younge

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 26th, 1953, Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 2, pp. 47-61. (Feb. 1953.)

an who really did not know what to make of a given il if he could classify it correctly. He had not then to g his heel into the ground and say, for instance, " That all right for two tons."

Mr. K. L. NASH congratulated Mr. Capper on his ost readable paper and on his elegant model.

On page 51 Mr. Capper had referred to the shear ength of soils, and in Fig. 5 he showed a Mohr circle d strength envelope for a $c - \varnothing$ material. In the xt it was stated that AR represented the plane of pture, and a plane parallel to AR was drawn on the set " sample " diagram. It was also mentioned, on ge 52, that the failure planes in a clay were not clined at 45°, despite the fact that the envelope was rallel to the σ axis. Thus there would appear to be me fallacy in the construction, and it was to that int that Mr. Nash directed his comments.

From the geometry of Fig. 5 it was seen that the

ilure planes were inclined at $(45 + \frac{\varnothing}{2})^\circ$ to the major

rincipal plane. The angle $\varnothing$ found from measuring ie direction of those planes was in fact the true angle internal friction of the soil, as had been shown by kempton.* Hence, if the construction in Fig. 5 were be used, the stresses must be effective stresses and ie envelope the true friction, true cohesion, envelope.

To illustrate the point he gave the results of a few ndrained triaxial tests with pore pressure measure-ients made on a fine sand (having no true cohesion) at ing's College, London :—

When plotted with respect to effective stresses, however, tests 1 and 2 gave approximately the same circle, and the construction applied to that strength

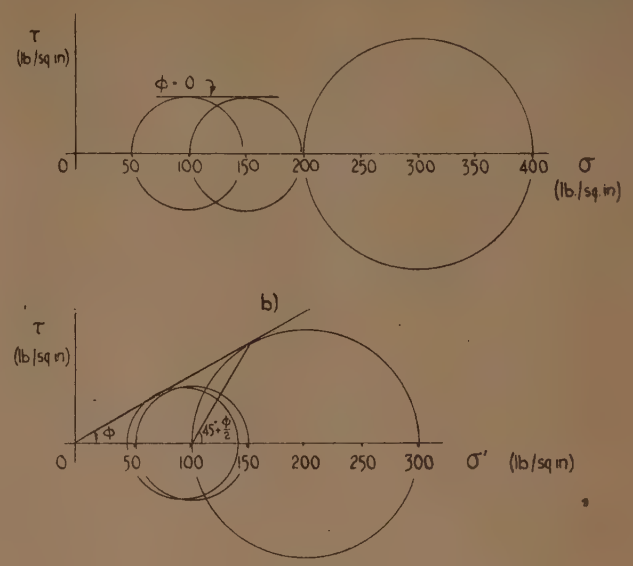


Fig. A

Mohr circle diagrams for undrained tests on Newburn sand.
a). In terms of total stresses
b). In terms of effective stresses

envelope gave the failure planes at about the right angle for all three tests.

If the construction were to be applied in Fig. 7, it must be to the $\varnothing_t$ and not the $\varnothing_d$ envelope. Otherwise one would get quite a wrong result.

Test No.	Consoln. pressure (lb./sq. in.	Porosity after consoln.	Stresses of failure (lb./sq. in.)					
			σ_3	$\sigma_1-\sigma_3$	σ_1	u_w	σ_1^1	σ_3^1
1	5	44.65	50	97.9	147.9	4.8	143.1	45.2
2	5	44.61	100	99.0	199.0	48.5	150.5	51.5
3	100	44.0	200	201.0	401.0	99.0	302.0	101.0

The first two samples were practically identical. Both ere consolidated by merely a small pressure. In the rst one, having consolidated the sample, the cell ressure was built up to 50 lb./sq. in., and in the second ne it was 100 lb./ sq. in., but both samples had prac-cally the same strength $(\sigma_1 - \sigma_3)$. In the first sample ie pore water pressure at failure u_w was 4.8, and in the econd one it was 48.5 lb./sq. in.

The third sample was consolidated to a slightly lower orosity, and it had very much greater strength than the rst two. That was shown in the Mohr circle diagrams.

When the results of the first two tests were plotted ith respect to total stresses, the strength envelope dicated, as for a clay (and for the same reasons), that iere was no friction at all, $\varnothing = 0$. Unless the effective resses could be increased (as in test No. 3) there could e no increase in strength. The direction of the failure lanes was not that given by joining the minor principal ress to the point of contact with that envelope. An nvelope to the three circles would be meaningless.

Dr. B. H. KNIGHT (Member) said that when he had heard Mr. Capper say in the course of his opening remarks that he would not offer anything new he had experienced a feeling of enormous relief, for there was a spate of new work.

Dr. Knight went on to ask if Mr. Capper considered there was any real sense in Young's modulus as applied to a soil. In the literature of the subject everybody was extremely cagey about Young's modulus for a soil ; and at various Research institutions he had asked what they understood by it, but he had never obtained a direct reply, always being told that work was on progress and was not yet ready for publication. That was rather wearisome. If Young's modulus changed all the time while a load was applied, obviously the assumption made by Boussinesq would fall and a lot of the graphs in the paper would not apply. So that we did not know where we were.

On page 60 of the paper there was a reference to the plate-bearing test, which Mr. Capper had suggested for measuring the load-bearing properties of a soil and, as he had said, it was a good " model " test, used by all sorts of people for finding out what loads they could put on to soils. But Dr. Knight confessed that he

*Skempton, A. W. A Study of the Geotechnical Properties some Post-Glacial Clays. GEOTECHNIQUE, I, 1, 7-22. June, 948.

had doubts as to the real value of a plate-bearing test, and he asked if Mr. Capper considered that the variations in soil mositure content, seasonal or otherwise, affected the results of such tests.

More important and more fundamental was the point concerning the conditions of loading in the test. Dr.

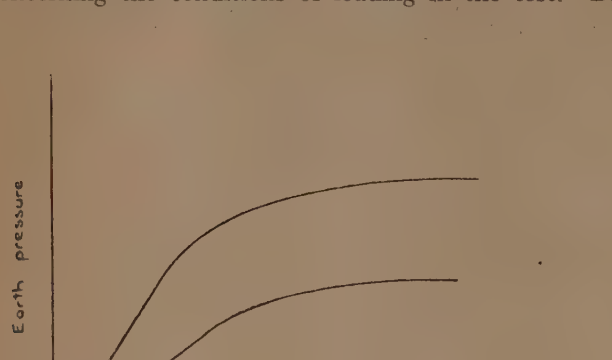


Fig. 1

Knight suggested that the rigidity of a steel plate of that kind would be comparable with that of (say) a concrete footing. Whilst admitting that roads and runways were not within the compass of the paper, although a runway was a structure of sorts, he asked

In thanking Mr. Capper for the extreme lucidity his paper, Dr. Knight said he would keep it by him a would use it in the future. Mr. Capper had rendered very great service to the Institution.

Professor A. L. L. BAKER (Vice-President) co gratulated Mr. Capper and thanked him for his compr hensive review of soil mechanics to date.

Commenting further on Mr. Capper's reference to t importance of achieving a balance between rigidity a flexibility, he recalled a problem of design which he h encountered some years ago in a country about 8,0 miles away. He and his colleagues had first consider a case one of them knew of a town hall which had be

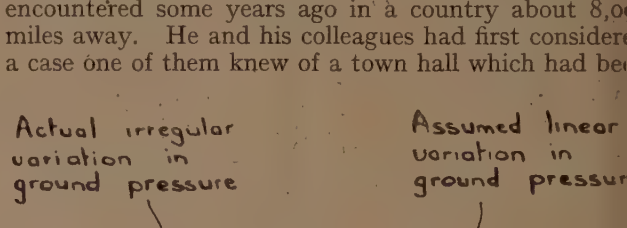


Fig. 3

designed for uniform earth pressure. They had work out the deflection corresponding to the very hi calculated bending moment of about 100 × 10^6 inch l and it came to more than 6 in., so either the town h must have cracked to pieces or else that high bendi

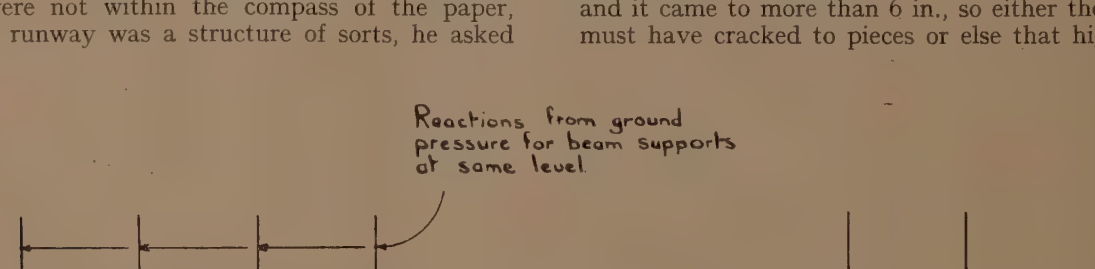

Reactions from ground pressure for beam supports at same level
Uniform ground pressure
a
Actual column loads minus reactions at a
b

Fig. 2

for the author's view as to whether the results of plate-bearing tests applied with a rigid steel plate would give any idea of what happened as compared with the results that would be obtained from a heavy roller with rubber tyres, a road lorry or anything else of that sort. It had always seemed to him that, when concerned with loads of that type, the results of plate-bearing tests made with a rigid steel plate were very risky, and he felt that Mr. Capper had put down something of a herring. He had put the question but had not answered it, and Dr. Knight asked for his ideas on the matter because the stress distribution was very complex in a plate-bearing test and it was difficult to simulate a rubber tyre. He (Dr. Knight) had very little use for the plate-bearing test as ordinarily carried out, for the reasons he had stated.

stress had never developed; he concluded that t latter was the true explanation.

So turning to their own problems they had consult all the vague soil mechanics data available at that ti and had considered the heterogeneous conditions of t sites on which they had to build, sites reckoned to ta only about three-quarters of a ton per sq. ft. They h to provide a 15-storey building and a 12-storey buildi both carrying very heavy live loads, and the proble was to avoid a very extravagant design. The essent information to be obtained from the soil mechani experts was contained in two load/settlement curv which were limiting curves for the site such as in Fig.

Taking the case of a beam of the raft, he said that if divided the load into two parts, the uniform grou pressure and the reactions which would occur, if

ssumed it to be the beam of a reversed floor supporting ne earth pressure and calculated the reactions in the ume way as in the case of a continuous beam on level ipports (see Fig. 2 (a)), and if we subtracted those eactions from the column loads we should get a set of rces as at Fig. 2 (b), in which the forces due to the eavier inside column loads in this case tended to deflect ne beam down as a whole. With the beam deflecting down, e had a variation in ground pressure, which was higher t the middle and less at the ends. The variation was regular, but if, however, we assumed that it varied in ccordance with a straight line law (see Fig. 3), we mplified matters very much indeed and we were not ery far out in the calculations, particularly if we orked between the limits of Fig. 1 ; and the interesting oint was that if we designed a raft with an appropriate

horizontal line through the point $Y = \frac{3}{8}$ in., and within the safe limits we obtained from the soil mechanics experts the building would take up no greater deflection than we had allowed, and therefore we should have effected that balance between flexibility and rigidity at which we aimed, and by providing maximum safe flexibility reduced the bending moments to a minimum.

Mr. F. L. Cassel (Member), after endorsing the expressions of appreciation made by previous speakers, said that in practice one very often came across results which did not fit easily the results we should expect from theory. As an example he referred to the determination of the shear strength of fissured clays, a frequent task, particularly in the London area. The shear strength was required to determine the ultimate

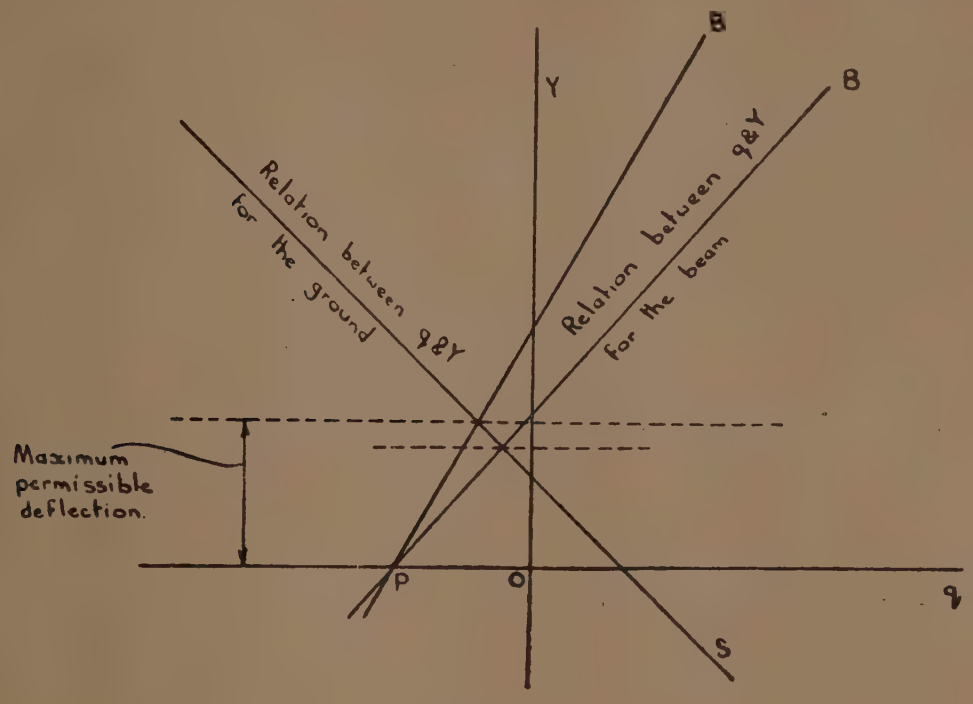


Fig. 4

lexibility, the bending moments, in spite of those very ide limits, were not very high.

Supposing we said that the raft for the building must e of such flexibility that under the worst conditions f live load—which in the case under discussion would e the heaviest live load concentrated over the soft round assumed in the middle half and no live load over he hard ground assumed over the outer quarters—the uilding must not deflect more than $\frac{3}{8}$-in., which was airly reasonable for a length of 75 ft. to 100 ft. to avoid racking. Then if qW (see Fig. 3), were the increase and lecrease over the uniform ground pressure and if y were he deflection of the beam, and if we plotted y against q or the beam (see Fig. 4, line B), the gradient of the line ould depend on the stiffness of the beam ; so that if e made the beam more flexible it would pass through he same point P and simply increase in gradient. If e said the deflection must not be greater than $\frac{3}{8}$-in., and f we plotted q against W for the soil, i.e., plotted the ariation of pressure in the ground against the difference n settlement of the ground (see Fig. 4, line S), then the oint at which the two lines intersected would be the esultant deflection. Then all we had to do was to nd out the gradient of the line B which would pass hrough the point of intersection of the line S with a

bearing capacity and eventually the permissible load. The outcome of triaxial compression tests should be theoretically, in accordance with Fig. 6 in the paper. If the clay were relatively soft and plastic, that was usually the case, but if it were near or below the plastic limit the results were often different and perplexing.

The apparent compression strength of a number of specimens from the same undisturbed core might be very erratic. Mr. Cassel illustrated that.

To prove that this was not due to an angle of shearing resistance, if such existed fortuitously, he said the same result would appear in such cases if all the tests were carried out at the same lateral pressure.

The cause was not unknown. In stiff fissured clays the soil was permeated by a network of fissures which became more pronounced when the soil dried out.

If a large number of those fissures happened to be in the direction of the largest shear stresses or near to it, failure along the plane of rupture shown in Fig. 5 would occur prematurely, before the full shear strength of the material was developed.

If the fissures were completely at random in the specimen, of if by chance they were all horizontal, no

such effect would occur. Between those two extremes, every intermediate position and strength might therefore appear in the test.

That that was so could be proved by completely re-moulding the samples without any change of water content. Then it would often become obvious that, contrary to the otherwise well-established rule that the shear strength of the re-moulded clay was less than that of the undisturbed sample, an increase of shear strength was found, as he had shown by the dotted lines on his sketches.

So far the facts were indisputable. He was letting that skeleton out of the cupboard before somebody did so in order to use it as a stick with which to beat the soil mechanics boys, particularly as that had happened at a previous meeting of the Institution.

The question arose : how should the results be interpreted for the assessment of the bearing capacity of such soil ? It was generally assumed that those fissures under a foundation, because of their random distribution in a large mass as opposed to a small test specimen, did not affect the bearing capacity.

If that were so, one would be justified in neglecting the lower values of shear strength and in basing the calculation on the stronger samples or the mean value of a number of them. He would not go so far as to refer to the strength of the re-moulded samples. But he believed that the method of averaging the higher results, particularly in a soil so well-known as the London clay, was justified and that, if an error were committed, we should still be on the safe side. He would be grateful to learn whether Mr. Capper would find that method acceptable, or which other he would recommend.

On the question of settlements, he said Mr. Capper had written that settlement records were not difficult to obtain. His own experience, however, was different. Mr. Capper might mean that it was not difficult to make the observations ; but unfortunately very few people cared to make them, particularly to start them at the right time, and therefore, particularly on smaller projects, he found it very difficult to obtain such records. He had never been able to obtain a settlement record which started at the beginning of the building operation and continued for about ten years.

Issuing a warning on the matter of site investigation, he said Mr. Capper had mentioned that that was usually entrusted to specialists, with good reason, as everybody who had studied the paper would agree. But Mr. Cassel urged that if a specialist were asked to do that work, his hands should not be unduly tied, as was the case only too often. Strict specifications were drawn up by Consulting Engineers and particularly by public authorities, telling the specialist exactly where he should take samples, what tests he should carry out and what he should not do, and all that without previous knowledge of what the conditions would be like. The result was very often that information which could be obtained was not obtained, and not seldom that the costs became much higher than they should be. He remembered a public tender involving nine borings and in which not less than a hundred consolidation tests were specified, work which would occupy about a year in a moderately equipped laboratory. Two were actually carried out. He suggested that the specialist should be acquainted with the problem and should decide what information he required and how to obtain it, at any rate within limits. He agreed that there might be exceptions to that procedure, particularly where the engineer was something of a specialist himself. But that was certainly not the rule to-day. He felt sure Mr. Capper would not object to what he had said on that matter.

Mr. Gower B. R. Pimm (Past President) derived real pleasure from the opportunity to add his meed of praise concerning Mr. Capper's work, particularly because some four years ago, when he had presented a paper on soil mechanics Mr. Capper undertook the almost impossible task of proposing a vote of thanks, and he would never cease to be grateful for the way Mr. Capper had proposed it.

In Mr. Capper's paper, at the bottom of the second column on page 47, it was stated :—

> " When comparing soils with other materials two essential difficulties arise, namely, the lack of homogeneity of natural soil, and the great variation of the properties of the soil with the water content. It may be largely because of these two uncertain factors that soil mechanics has for so long lagged behind other branches of engineering science."

In the first column on page 48 it was stated :—

> " Authorities on soil mechanics do not claim to provide a one-hundred-per-cent. correct answer to all our practical problems."

Mr. Pimm's only complaint was that Mr. Capper whilst admitting the great difference between soil and other materials and that there was not one-hundred-per-cent. accuracy, did not say what we should do about it.

Referring again to the occasion on which he had presented his own paper, he said he had described soil mechanics as a 1 : 2 : 4 mixture—one part finely ground theory, two parts eyewash and four parts self-deception.

Mr. Capper had dealt in a brilliant way with the finely ground theory and had done a great deal to clear up the aggregate, but one wished he could do a good deal more.

On the same occasion a speaker—whose identity he would not disclose, but to whom he would refer as " Mr. Johnson."—had said that before we applied test results to practice we must apply the " Johnson co-efficient of so-what-ness." He had not meant a co-efficient in the literal sense, but there was an element of truth in the suggestion that we wanted a co-efficient of so-what-ness between laboratory results and their application in practice.

The works of the greatest authorities were full of warnings of the extent of variations that could occur in soils, even in different specimens from the same sample and Mr. Pimm quoted two such warnings from Terzaghi's James Forrest Lecture—" A New Chapter in Engineering Science " (JOURNAL I.C.E. June, 1939). The first was illustrated by a test which Terzaghi made to ascertain the variation of moisture content in the London clay. There was a variation from 18 to 35 per cent. in the moisture content at different parts of a sample which before the test was made, had been considered uniform.

In the same lecture Terzaghi made the following more general pronouncement on the same question.

> " Due to the universal absence of homogeneity, the essential pre-requisite for selecting representative samples consists in securing complete data on the variation of at least one property of the soil along several vertical lines."

There were reliable methods of obtaining an absolutely continuous record of that kind, thereby avoiding the

uncertainty of relying upon tests on samples taken at various intervals.

In "Soil Mechanics in Engineering Practice," by Terzaghi and Peck, there were innumerable warnings that there is something lacking between the results of laboratory tests, as usually carried out, and their application to design, and Mr. Pimm quoted two of these :—

(p. 242) "If the test boring records indicate that a bed of clay contains partings of sand or silt, the engineer is commonly unable to find out whether or not these partings are continuous. In such cases the theory of consolidation can be used only for determining an upper and a lower limiting value for the rate of settlement. The real rate remains unknown until it is observed."

(p. 306) "If the profile of the subsoil of a proposed structure is erratic, undisturbed sample borings and elaborate soil investigations are entirely out of place. Information of far more practical value can be obtained by means of numerous sub-surface soundings supplemented by exploratory drill holes."

Mr. Pimm exhibited a slide which reinforced what Mr. Cassel had said. It showed, by means of Mohr circles, the results of a complete series of tests in the investigation of a site by one of the leading firms of soil mechanics specialists. Mr. Capper had drawn only two Mohr circles in any of his illustrations. What happened if more were drawn? The slide threw considerable light on this, and showed that in only eight out of the eighteen tests could anything approaching a common tangent be drawn.

Finally, Mr. Pimm expressed his complete agreement with Mr. Donovan Lee concerning Housel's perimeter-shear theory, which had been sadly neglected, although of great value.

He hoped that on some future occasion Mr. Capper would tell us how to assess the probable accuracy of laboratory tests, and what we should do about it when it falls too far short of one-hundred-per-cent.

Professor W. Fisher Cassie (Member) said that as he had the privilege of collaborating frequently with Mr. Capper he would leave the posing of questions to other members of the audience. He then supported other speakers in their remarks on the usefulness of Housel's perimeter-shear method and described an example which occurred in practice where this theory had led to predictions which were confirmed by a full-scale test.

Mr. D. J. Maclean, referring to the quotation of the remarks of Terzaghi about the need in the further development of soil mechanics for effort to be concentrated mainly on full-scale investigations, thought there was a possible danger of misunderstanding Terzaghi's remarks when considered out of their context. These remarks were made in 1948 at the Second International Conference on Soil Mechanics and Foundation Engineering, where many papers were highly theoretical and of little practical value. It was probable that Terzaghi hoped, by making a strong plea for more full-scale work, to sway the future trend in soil mechanics research towards a better balance between theoretical and practical work.

Mr. Maclean did not think that practical site investigations dissociated from laboratory work were of any more value than laboratory investigations dissociated from practice. What was needed was very careful correlation of the behaviour of soils in practice with laboratory studies of their properties; without laboratory investigations it was not possible to interpret correctly what happened in practice.

Commenting on a graph of the moisture distribution in ground which Mr. Gower Pimm had demonstrated to illustrate the irregular variation of moisture content with depth and therefore the little practical value of recognised soil mechanics tests, investigations showed that if instead the pore-water pressure of the soil was measured, a uniform relationship would have been obtained. The fact was that moisture content was one of the least significant measurements that could be made on soil. He thought therefore that the conclusion to be drawn was not that soil mechanics tests were of no practical value but that further research was needed to devise tests which have greater significance in interpreting the practical behaviour of soils.

Mr. Maclean wondered to what extent members of the Institution were concerned with building abroad, since no reference had been made to soils overseas. Engineers were often faced with considerable difficulties through the lack of knowledge of foreign soils, as for example in Africa, that usually differed considerably from those occurring in this country, and also through the very big effect of climate. For example, in South Africa research was being conducted into the swelling of soils which caused serious damage to buildings. Much more knowledge was needed of foreign soils to assist engineers who were carrying out works overseas.

A serious weakness in our present knowledge of the application of soil mechanics to the foundations of structures was the lack of practical verification of stress distributions in the ground. The Road Research Laboratory was investigating this problem on a small scale but much more work was required on this problem.

Mr. Maclean referred to the comment by Dr. Knight that he could not obtain a direct reply from the various research institutions concerning the meaning of Young's modulus for soil. If, as he imagined, he included the Road Research Laboratory amongst those institutions then it might be remarked that the values quoted in Dr. Knight's book, "Soil Mechanics for Civil Engineers," 1st Edition, pages 62 and 63, Tables 48, 49 and 50, that were obtained from measurements made at the Laboratory several years ago, had been confirmed recently by resonant vibration measurements at audio frequencies. Such values applied only to very small strains and to the conditions of loading employed in the tests—they should never be used for static loading conditions. It was realised that Young's modulus varied with the loading conditions, particularly with the rate of loading, and this problem was being studied in a number of laboratories including the Road Research Laboratory. For roads, measurements were being made of the waves of stress both in the soil sub-grade and in the road materials when vehicles travel along the road at various speeds. By combining these results with those of Young's modulus at various rates of loading, it was hoped that a comparison would be possible between the measured stress distribution and those computed by the various available elastic theories. It was of interest to record that Sven G. Bergstrom, of the Swedish Cement and Concrete Association, had shown that elastic theories applied to the case of a loaded concrete road structure.

Finally, Mr. Maclean added his thanks to Mr. Capper for a very interesting and valuable paper.

Mr. Capper replied to the discussion.

On the Rankine and Bell theories, he suggested that Mr. Donovan Lee was perhaps under a slight mis-

apprehension. He had mentioned in the paper that as regards the bearing pressure on foundations those theories were not in accordance with experimental results. When it came to lateral pressures on retaining walls it was quite a different story and the remarks which Mr. Donovan Lee had read from his book applied quite well.

With regard to the stress distribution, Boussinesq's theory was based on the ideal condition of a concentrated load on the surface of a semi-infinite elastic solid. By integration we could get the effect of distributed load. It was not claimed that that gave the true stress distribution, but in the present state of knowledge it was the best we could do ; experiments were proceeding and no doubt we should get something better in due course.

Coming to Mr. Donovan Lee's reference to the contact pressure distribution being a maximum at the edges on cohesive soil, he said it was correct that, as we took the footing down below ground level, the irregularity tended to be smoothed out. But he had also pointed out that, when it came to the pressure at considerable depth, for settlement purposes or in connection with the stability of the foundation as a whole, the pressure distribution mattered less than might appear at first sight.

As to Housel's perimeter-shear theory, he said that in the book by Terzaghi and Peck, the usual Housel formula, which was quoted in the paper, was established, but by a slightly different reasoning from that originally put forward by Housel himself. He was glad to have heard examples in confirmation of that theory.

In regard to unconfined soil tests, he believed it had been the experience that the compression test tended to give low values.

He had nothing available concerning the work of Caquot, but he would try to look up something with regard to it.

Dealing with Mr. King's reference to the importance of the study of geology, he said he was absolutely with him and he had mentioned at the end of the paper that we wanted more information about the average soil properties to be expected from soils as classified according to the Code of Practice " Site Investigations, Table I."

Some useful information was given by Mr. Nash by way of amplification on the question of the true angle of friction compared with the angle of shearing resistance which we obtained ordinarily from the shear tests. Mr. Capper said he had endeavoured to explain it briefly in the paper, but it was rather difficult to do so adequately in the limited space. Incidentally, in a book written by Mr. Nash it was explained very clearly, again in quite a small space.

Replying to Dr. Knight, he said he did not think that Young's modulus of soil was of much direct use even if we could get it, because even in an ideal elastic medium, if we wanted the load/settlement relation, something much more complicated was required than the simple elastic modulus. But we did want the load/settlement relation as obtainable from suitable tests ; and we wanted a good deal more information about it because he believed there was a considerable scale effect involved, and that the load/settlement relation derived from the small scale test, even when the conditions were apparently the same, might differ considerably from that obtained on the large scale.

As to the plate-bearing test and its application to runways, and the comparison with wheel loads, he did not think there was any direct comparison. But if the plate-bearing test were used to determine a kind of modulus for the soil, which could be properly co-related to the behaviour of a runway or an ordinary foundation, it might be of great value ; but it should not be used directly.

Some very interesting information was given by Professor Baker about his method of designing footings, where the behaviour of the soil and the flexibility of the raft was taken into account. It rather indicated that we wanted some more reliable methods of obtaining soil information data. He was interested in Professor Baker's two curves showing an upper and a lower limit, for he felt that in work on soil, where there was so much variation, there were many problems in regard to which we must expect to work to such limits instead of an exact figure.

The information given by Mr. Cassel on fissured clays was also very interesting. The answer, in the case of fissured clay, was that we had been accustomed to specimens for the ordinary unconfined and triaxial tests, shear boxes, and so on, which were too small. Many people, nowadays, however, were using 4 in. diameter triaxial specimens, and shear boxes 12 in. square had been used, and it was likely that we should have to get accustomed to much bigger apparatus in order to deal with more representative samples. We must bear in mind that laboratory and plate-bearing tests, and so on, were model tests ; the use of models had been developed very considerably in mechanical and hydraulic engineering, and the scale effects had been known, and we had similar effects to be considered in soils. There was room for great improvement in that direction.

He agreed with Mr. Cassel that every encouragement should be given to the taking of observations of settlement ; and, of course, if they were to be of any use they must be started at the right time.

As to the conditions imposed by engineers on the soil specialists, he felt that the position would right itself when more engineers got to know the value of soil mechanics and appreciated what was required in site investigations ; they would then give the soil specialists more freedom or, if they had to impose restrictions, those restrictions would be more reasonable.

Coming to the interesting suggestions made by Mr. Gower Pimm, he agreed that all the theory and the findings from laboratory work had to be applied in the light of experience and common sense. We must remember that soil mechanics was in its infancy as compared with many other branches of engineering, and he did not think we could claim to know all the answers even in regard to reinforced concrete or any other engineering materials.

After thanking Professor Fisher Cassie for having given more examples of the usefulness of the Housel theory, Mr. Capper came to Mr. Maclean's remarks in favour of correlation with laboratory studies, which served to emphasise the point made in the paper that by the co-ordination of the work of the theorists and researchers and of the practical engineers and contractors in the field we should make progress.

Finally, he said the remarks by Mr. Maclean on the swelling of soils, and its effects, were also of interest, for that factor was very important in some soils abroad. It was impossible, however, to include reference to it in a paper of the kind he had presented.

Mr. Capper thanked the meeting for the manner in which his paper had been received and for the interesting discussion which had followed.

At the conclusion of the meeting, the President expressed appreciation of the way in which Mr. Capper had replied to the discussion.

Institution Notices and Proceedings

ANNUAL GENERAL MEETING

The Annual General Meeting of the Institution of ructural Engineers was held at the Royal Technical llege, Glasgow, on Thursday, May 21st, 1953, at 30 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., .I.Struct.E. (President), in the Chair.

The Secretary (Major R. F. Maitland, O.B.E.), read ıe notice convening the meeting.

The minutes of the Annual General Meeting held on ay 22nd, 1952, as published in THE STRUCTURAL NGINEER, July, 1952, were taken as read and were nfirmed and signed.

Mr. A. H. Ley moved the adoption of the Sessional eport of the Council and the accounts for the financial ear, 1952. Mr. A. P. Mason (Past Chairman Midland unties Branch) seconded the motion, which was rried unanimously.

Mr. A. H. Robb (Past Chairman Yorkshire Branch) roposed the re-election of Messrs. James Meston & Co., hartered Accountants, as Auditors for the ensuing ear. Mr. Walter C. Andrews (Past President) seconded ıe motion, which was carried unanimously.

The Secretary then read the report of the Scrutineers ı the ballot for the election of President, the Honorary fficers and the ordinary members of Council for the ession 1953-54, as follows :—

To the Council of the Institution of Structural ngineers :

entlemen,

We, the undersigned, report that at the request of the resident we have duly carried out the duties of Scru-neers of the Ballot for the election of Honorary Officers ıd Council for the Session 1953-1954, and we report ccordingly as follows :

We received 732 Ballot Papers of which we rejected 5 as wholly spoiled and 28 as partly spoiled. We have ttached a separate sheet showing the number of votes eceived by each candidate.

We declare the result of the Ballot to be as follows :—

ELECTED

resident : Lt.-Col. R. F. Galbraith, R.E., M.C., B.Sc.(Eng.), A.M.I.C.E.

ice-Presidents : Dr. S. B. Hamilton, M.Sc., Ph.D., A.R.C.S., M.I.C.E.
Mr. S. Vaughan, B.Sc., M.I.C.E., A.C.G.I.
Mr. J. Guthrie Brown, M.I.C.E.
Professor A. G. Pugsley, O.B.E., D.Sc.,(Eng.), F.R.S., M.I.C.E., F.R.Ae.S.
Professor A. L. L. Baker, B.Sc.(Tech.), M.I.C.E.
Mr. G. S. McDonald, M.I.C.E.

on. Treasurer : Mr. L. E. Kent, B.Sc.(Eng.), M.I.C.E.

on. Secretary : Mr. J. Singleton-Green, M.Sc., A.M.I.C.E., A.M.I.Mech.E.

on. Librarian : Mr. Wallace A. Evans.

on. Editor : Mr. W. H. Woodcock, F.C.S.

on. Curator : Mr. F. R. Bullen, B.Sc.(Eng.), M.I.C.E.

The above are all elected for one year.

LECTED AS ORDINARY MEMBERS OF COUNCIL (LONDON)

Mr. D. H. New, B.Sc.(Eng.), A.C.G.I., D.I.C., M.I.C.E., A.M.I.Mech.E.
Mr. John Mason, B.A.(Cantab.), A.M.I.C.E.
Mr. T. Bedford, M.I.C.E., M.I.Mech.E.

The above are elected for three years.

ELECTED AS ORDINARY MEMBER OF COUNCIL (COUNTRY)

Mr. A. P. Mason, B.Sc., M.I.C.E.

The above is elected for three years.

ELECTED AS ASSOCIATE-MEMBER OF COUNCIL (LONDON)

Mr. D. T. Williams.

The above is elected for three years.

ELECTED AS ASSOCIATE-MEMBER OF COUNCIL (COUNTRY)

Mr. H. V. Hill, M.Sc., A.M.I.C.E., M.Inst.W.

The above is elected for three years.

We are, Gentlemen,
Yours faithfully,
(signed) H. BROMPTON
D. M. ELEY, Lt.-Col.
S. M. REISSER
C. B. BROWN
(Scrutineers)."

On a motion by the President, a vote of thanks was unanimously passed to the scrutineers.

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, May 28th, 1953, at 5 p.m., Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership ?

STUDENTS

BENFORD, Edward John, of Birmingham.
SMITH, Maven, of Manchester.
WARD, Ralph Northern, of Bulawayo, Southern Rhodesia

GRADUATES

ABBAS, Abdel Moneim, of London.
BEESON, Alan Spencer, of Isleworth, Middlesex.
BOWMAN, Charles William, A.M.I.C.E., of Edinburgh.
BULLOCK, Leslie, A.M.I.C.E., of London.
BUXTON, Leslie Gordon, of Nottingham.
COGGON, Harry Douglas, B.Sc.(Eng.) London, of Derby.
CROKER, Arnold Meyrick, of Twickenham, Middlesex.
CRONIN, Denis Paschal, B.E. Cork, of Cork, Ireland.
DICKSON, James, of Burton-on-Trent, Staffs.
DOORLY, Dermot Kieran, of London.
EATON, Peter Harrington, D.F.M., of Leicester.
ELDRIDGE, James Gordon, B.Sc.(Eng.), A.M.I.C.E. London, of Tunbridge Wells, Kent.
GARRITY, Albert Francis, of Stockport, Cheshire.
GOODIER, Roger Ewart, of Preston, Lancs.
GOULD, Ian Robertson, of Hale, Cheshire.
HOLMES, Geoffrey, of Gildersome, nr. Leeds.
JOSHIRAO, Chintaman Moreshwar, of Poona, India.
KAFAROWSKI, Zygmunt, B.Sc.(Eng.) London, A.C.G.I., of London.
KAPKA, Jerzy, of London.

KARNIK, Arvind Vasant, B.E.(Civil) Bombay, of Birmingham.
KHAN, Rao Abdul Hameed, of Bombay, India.
KOH BENG SWEE, of Singapore.
KOMORNICKI, Wladyslaw Stanislaw, of London.
KWIECINSKI, Franciszek, of London.
LEROI, Jules Anne, of Christchurch, New Zealand.
MORGAN, Albert Edward, of St. Helens, Lancs.
MUNRO, John, B.Sc.(Civil) Glasgow, of Beckenham, Kent.
NARAYANA, K. Yogna, of Madras, India.
O'CALLAGHAN, John Francis, B.E., N.U.I., of Dublin.
PATWARDHAN, Sadashiv Vasudeo, B.E. (Civil) Bombay, of Ratnagiri, India.
PEATFIELD, Derek Edward, of Welling, Kent.
RAMSDEN, Jack Kenneth, of Leeds.
ROLFE, John Millar, B.Sc.(Civil) Cape Town, of Que Que, Southern Rhodesia.
SALMON, Norman Derek, B.Sc.(Civil) Rand, of Pretoria, South Africa.
SLEE, John Penry, B.Sc(Eng.) London, of London.
SMITH, Ronald Walter, of Newcastle-upon-Tyne.
STERN, Walter Mortiz, B.Sc.(Eng.) Rand, of Pretoria, South Africa.
TREICHEL, Jan, of London.
TREZISE, Bernard, of Bristol.
VIELROSE, Witold Leon Ryszard, of London.

ASSOCIATE-MEMBERS

ALTERMAN, Israel, of Tel-Aviv, Israel.
ARMSTRONG, George Herbert, of Edinburgh.
ASZ, Leon, of Salisbury, Southern Rhodesia.
ATKINS, George Frederick, of Surbiton, Surrey.
BAIRD, James, of Glasgow.
BELCHER, Charles Duncan, of Hampton Hill, Middlesex.
BOXALL, Charles Alfred, of Morden, Surrey.
CAWOOD, Peter Ireland, of Durban, South Africa.
CHANDRANGSU, Sirilakn, D.I.C. London, of London.
DERRINGTON, John Anthony, B.Sc.(Eng.) London, A.M.I.C.E., of Twickenham, Middlesex.
DUTTON, Charles Norman, of Morden, Surrey.
EVANS, George Harold, B.Sc.(Eng.) London, of Berkhamsted, Herts.
FIELD, James, B.Sc.(Eng.) London, of London.
GAMSE, Joel David Harold, B.Sc.(Eng.) London, D.I.C., of Edgware, Middlesex.
GLAZE, Albert Gordon, B.Sc.(Tech.) Manchester, of London.
GRAY, Robert, of Burgess Hill, Sussex.
GUDGEON, Donald Keith, of Bromley, Kent.
HAYNES, Alan, M.A.Cantab., A.M.I.C.E., of Tamale, Gold Coast, British West Africa.
JAAP, John Hunter, of Durban, South Africa.
KANVINDE, Sitakant Shantaram, B.E.(Civil) Bombay, of Bombay, India.
LEWKOWICZ, Janusz Jozef, A.M.I.C.E., of London.
MAUNDER, Augustine Hamilton, B.Sc. New Zealand, of Wellington, New Zealand.
NEWTON, David Wilson, A.F.C., of Johannesburg, South Africa.
OLIVER, Carl Richard, of Darlington, Co. Durham.
PECKHAM, Donald, of South Ruislip, Middlesex.
POMEROY, Albert Frederick, of London.
ROSENAK, Sidney, B.Sc.(Eng.) London, of London.
SCHOU, Jorgen Arthur Kjeld, of Nairobi, Kenya.
SRINIVASAN, Kilnagar Vedachala Iyer, of Madras, India.
STOTT, Harold Welch. B.Sc.(Eng.) London, of London.
SZCZERBANIEWICZ-STEPHENS, Stanislaw Jozef Michal, of Enfield, Middlesex.
SZPILEWICZ, Jan, of London.

MEMBERS

MAIS, Colonel Alan Raymond, O.B.E., T.D., D.L., of London.
PEARCE-PERCY, Thomas Walker, of Melbourne, Australia.
WELLISCH, Walter, of Durban, South Africa.

TRANSFERS

Students to Graduates

ABRAHAMS, Frank Dudley, of Birmingham.
AGER, John Stanley, of London.
ALLANSON, Brian Melvyn, of Bolton, Lancs.
ASTILL, Alan Walter, of Nottingham.
BAKALA, Eugeniusz, of London.
BENG WEE GIAP, of Singapore.
BEZUIDENHOUT, Willem Jacobus, of Germiston, South Africa.
BOTING, Henk, of Bloemfontein, O.F.S., South Africa.
BROWN, Geoffrey Hewitt, of Winsford, Cheshire.
CHAN WENG CHIU, of Singapore.
CLARK, Peter Jerome, of Birmingham.
COXHEAD, Peter Leonard, of Beckenham, Kent.
CRAGG, Wilfred Shipham, of Mowbray, Cape, South Africa.
DAINES, Alfred Lionel, of Carlisle, Cumberland.
FOLLY, Eric John, of Johannesburg, South Africa.
HOMER, Albert Leslie, of Bromsgrove, Worcs.
KNOTT, Stanley William, of Manchester.
KYCIA, Serafin, of London.
LANGHAM, Bernard Trevor, of Isleworth, Middlesex.
MILLIGAN, William Ramsay, of Sale, nr. Manchester.
PURVER, Gordon Douglas, of Coulsdon, Surrey.
RAWNSLEY, Peter Michael, of Bradford, Yorks.
READER, David, of London.
REFFITT, Peter, of Harrogate, Yorks.
SHERLOCK, William Albert, of Cardiff.
STANLEY, Brian William, of Stockton Heath, nr Warrington, Lancs.
STONE, Geoffrey Jones, of Trowbridge, Wilts.
TAYLOR, Phillip Austin, of London.
TONGE, Brian Yardley, of Bolton, Lancs.
WILSON, Russell, of Berea, Johannesburg, South Africa.
WRIGHT, Frank, of Milford, nr. Derby.

Graduates to Associate-Members

ARD, Donald Leonard, of Weymouth.
ARMSTRONG, John Bryan, of West Hartlepool, Co Durham.
AUSDEN, Alistair Charles, of Bushey, Herts.
AUSTIN, Michael, of Rickmansworth, Herts.
BAGULEY, Maurice Grant, of Bexleyheath, Kent.
BAKER, Alexander John, of London.
BAKER, Gordon Arthur Henry, of London.
BANFIELD, Arthur Reading, of London.
BIRCH, Norman, of Salford, Lancs.
BRAY, William Albert, of Stoke-on-Trent.
BRILL, Herbert, B.Sc.(Eng.) London, of London.
BROOKE, Jack, of London.
BURGE, Stuart, of London.
CHENHALL, Raymond Leslie, of Bristol.
COLLINS, Peter Courtney, of Saltdean, Sussex.
COLQUHOUN, William Alexander Smart, of Larkhall Lanarkshire.
DANAHER, Martin Denis, B.E.(Civil) New Zealand, of Wellington, New Zealand.
DAVIDSON, Ronald Frank, of Manchester.
DUNN, Allan George, of Coulsdon, Surrey.
EASTON, Edgar, of London.
GRADY, Jack, of Harrow, Middlesex.
GREEN, Edward Ellis, of Thorpe, Norwich.
HALL, Kenneth James, of London.

MER, Clifford, of Oldham, Lancs.
RVEY, Peter Lawrence, of London.
PPLEWHITE, Eric Anthony, B.Sc.(Eng.) London, of London.
GGINSON, Leonard, of East Kirkby, Notts.
DGKINSON, Arthur, B.Sc.(Tech.) Manchester, of Manchester.
RRIDGE, James Frederick, of Bolton, Lancs.
MPHREYS, Robert, of Middlesbrough, Yorks.
ACS, Basil, B.Sc.(Eng.) Rand, of London.
ARKAR, Meghashyam Shantaram, A.R.I.B.A., of Dadar, Bombay, India.
NSDALL, Kenneth Charles, of London.
VESEY, Keith Haydn, M.Sc.(Civil) Leeds, B.Sc.(Hons.), of Bo'ness, West Lothian, Scotland.
DLIN, Sydney, of Johannesburg, South Africa.
WTON, Christopher John Broughton, of Thornton Heath, Surrey.
CHOLSON, Thomas Hedley, B.Sc.(Civil) Durham, of Newcastle upon Tyne.
TES, Ronald, of Bridlington, Yorks.
MEARA, Brian Patrick, of Brighton, Sussex.
RISH, Jack Leonard James, of London.
XTON, Ian Hamilton, M.A.Cantab., of Leeds, Yorks.
RTER, Arthur James, B.Sc.(Tech.) Manchester, of Prestwich, Manchester.
ICE, Selwyn Lionel, B.Sc.(Civil) Rand, of Pretoria, South Africa.
THWELL, Leslie Herbert, of Eccles, Nr. Manchester.
JSSON, Robert George, of St. Pauls Cray, Kent.
AIKH, Mohammad Abdul Wahab, B.Sc., B.E.(Civil) Bombay, of Karachi, Pakistan.
NGH, Jattan, of Gorakhpur, India.
OWIKOWSKI, Leszek, B.A., of London.
EPHEN, Lawrence Ian, of London.
EWART, James Campbell, B.Sc.(Civil) Glasgow, of Glasgow.
RAPOREWALLA, Jalèjer Jehangir, B.E.(Civil) Bombay, of Bombay, India.
AYLOR, James Edward (Jnr.), of Birmingham.
EMPLE, Allan, of Middlesbrough, Yorks.
HORNTON, Paul Rosslyn, of London.
AO Hsien Hwa, of Singapore.
ESTALL, John Butterworth, B.Sc.(Tech.) Manchester, of Manchester.
ILLIAMS, Harry, A.M.I.C.E., of Timperley, Cheshire.
OODS, Malcolm Derek, of Salford, Lancs.
OOLNOUGH, Edwin Arthur, B.Sc.(Tech.) Manchester, of Kenton, Harrow, Middlesex.

Associate-Members to Members

ELL, Frank Victor Moore, M.Eng. Liverpool, M.I.C.E., of Llandarcy, Neath, Glamorgan.
UMBLETON, Winston Philip, of London.
ANE, Thomas, of Baillieston, Lanarkshire.
ATTHEWS, Denis Dearman, M.A. Oxon., D.Eng., M.Sc.(Eng.) London, A.M.I.C.E., of Manchester.

Members to Retired Members

UTLER, Frederick Arthur Leslie, L.R.I.B.A., of Bembridge, Isle of Wight.
AMPTON, Major Matthew Sommerville, of Waltham Chase, nr. Southampton.

OBITUARY

The Council regrets to announce the death of The EV. FRANK HAY GILLINGHAM (Honorary Associate); HARLES SEVER BELL, JAMES CLEMENT HUGHES, JOHN OYD LAWSON, JOHN DUNCAN MACLEOD, RAPHAEL UGENE TROCME, SIR NESS WADIA, GUTHLAC WILSON, CHARLIE ROLAND WOODS (Members); STEWART MCINTYRE, GEORGE MALTBY, ARTHUR EDWARD WILLIAMS (Retired Members); CHARLES AUGUSTUS ROPES ESLICK (Associate).

EXAMINATIONS—JULY, 1953

The Examinations of the Institution will next be held at Centres in the United Kingdom and overseas on July 14th and 15th (Graduateship), and July 16th and 17th (Associate Membership).

REPRESENTATION

The Council has made the following nominations of members to represent the Institution :—

MINISTRY OF EDUCATION—Joint Committee on National Certificate and Diplomas in Building
Lt.-Col. G. W. Kirkland, M.B.E. (Member of Council)

MINISTRY OF WORKS—Building Apprenticeship Training Council
Mr. D. A. G. Reid (Associate-Member)—re-appointment.

CODES OF PRACTICE COMMITTEES

The Council has approved the formation of the following Code Drafting Committees :—

SUB-COMMITTEE G—PRESTRESSED CONCRETE

CONSTITUTION

	Nominated by
Dr. F. G. Thomas (Chairman)	Department of Scientific and Industrial Research
Dr. P. W. Abeles	Institution of Structural Engineers
Prof. A. L. L. Baker	" " "
Mr. J. Cuerel	Reinforced Concrete Association
Mr. P. Cutbush	British Standards Institution
Prof. R. H. Evans	Institution of Civil Engineers
Mr. W. A. Evans	Institution of Structural Engineers
Dr. Oscar Faber	" " "
Mr. F. L. Felgate	Royal Institute of British Architects
Mr. C. W. Glover	Institution of Structural Engineers
Mr. G. J. Gumersall	London County Council
Dr. K. Hajnal-Konyi	Institution of Structural Engineers
Mr. A. J. Harris	Institution of Structural Engineers
Mr. A. D. Holland	Ministry of Transport
Mr. Donovan Lee	Institution of Structural Engineers
Mr. A. S. Meanley	Institution of Municipal Engineers
Mr. D. H. New	Institution of Structural Engineers
Mr. J. Singleton-Green	" " "
Mr. T. G. Wakley	London County Council
Mr. F. Walley	Ministry of Works
Prof. F. Webster	Prestressed Concrete Development Group
Mr. A. W. Hill	Cement and Concrete Association

SUB-COMMITTEE H—PRECAST FACTORY-MADE CONCRETE

CONSTITUTION

	Nominated by
Dr. Oscar Faber (Chairman)	Institution of Structural Engineers
Mr. M. C. G. Ayre	London County Council
Mr. P. G. Bowie	Cement and Concrete Association
Mr. P. Cutbush	British Standards Institution
Mr. F. L. Felgate	Royal Institution of British Architects
Mr. W. J. Haines	British Cast Concrete Federation

Mr. A. E. Hewitt	Ministry of Works
Mr. A. F. Holt	Institution of Structural Engineers
Mr. O. J. Masterman	,, ,, ,,
Mr. A. S. Meanley	Institution of Municipal Engineers
Dr. L. J. Murdock	Reinforced Concrete Association
Mr. W. Hunter Rose	Institution of Structural Engineers
Col. W. B. Sykes	British Cast Concrete Federation
Mr. T. Whitaker	Department of Scientific and Industrial Research
Mr. S. Preston	Cast Stone and Concrete Federation

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees of Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List " A " provide at least two years of instruction in Theory of Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List " A "

Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Salford, Royal Technical College.
South-West Essex Technical College, Walthamstow, E.17.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Plymouth and Devonport Technical College.
Preston, Herris Institute.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.

Students are advised to take the organised courses in Structural Engineering where these are available.

YEAR BOOK AND LIST OF MEMBERS

The Year Book and List of Members for 1953 will go to press in July, for publication in October, when a copy will be sent to all members.

Members are requested to inform the Secretary of any alterations in titles, degrees or addresses, which have not already been notified, by July 13th, in order that such amendments may be included in the new edition.

DRURY MEDAL AWARD

The fourth competition for the above award will take place in 1953. The subject is the design of the structure of a new factory building. The material of construction is entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of structural arrangement. Economy in the use of steel is an important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary ; envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st 1953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely e work of the competitor, the documents submitted all be countersigned by a corporate member of the stitution or, failing this, shall be accompanied by a claration on a prescribed form signed by the candidate the presence of a Justice of the Peace or a Commis-oner for Oaths.

MACLACHLAN LECTURE COMPETITION

The closing date for the receipt of entries for the acLachlan Lecture Competition is Wednesday, March st, 1954. Particulars of the competition are as llows :

1. The Institution of Structural Engineers shall stitute a written lecture to be known as the MacLachlan ecture, and to be held annually.

2. The subject of the Lecture may be on any aspect of tructural Engineering as long as in every second year ie subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture iall be confined to Associate Members of the Institution, ho are under the age of 32 years.

4. All papers entered for the competition shall be ibmitted to assessors to be appointed by the Council f the Institution, and all such papers (including the rize-winning Lecture) shall be available for publication i the Journal of the Institution at the discretion of the ouncil.

5. No paper submitted shall have been published or ead elsewhere.

6. The winner of the competition shall be required to resent the Lecture to a meeting of the Institution at hich he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy f ranking second in merit he will receive a consolation ward of £5.

8. In the event of there being no winner of the compe-ition in any one or more years, whether because no ecture submitted is considered to be of sufficient merit o warrant award, or for any other reason, the Institution hall transfer the above sums to the Research Fund of he Institution.

PARTICULARS OF THE COMPETITION FOR 1954

1. The MacLachlan Lecture will be given at a meeting f the Institution to be arranged towards the end of 1954.

2. The subject of the Lecture shall be on any aspect f structural engineering.

3. The work should be submitted as the script of a ecture which the author, if successful in the compe-ition, will deliver before an audience in the course of bout one hour. The development of mathematical ormulæ and detailed calculations should be avoided s far as possible in the text ; if they are essential they hould be embodied in appendices. Photographs, rawings, graphs, etc., which would appear as illustra-ions to the lecture in published form, should accompany he script. If additional illustrations would be shown s slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and hould be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the nstitution is Wednesday, March 31st, 1954.

RESEARCH AWARDS

The Council has instituted a Research Prize Fund, rom which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;

(b) studies of historical or statistical records ;

(c) improvements in principles or methods of construction ;

(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;

(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;

(b) the value of the paper in advancing the science and art of structural engineering ;

(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1952, and September, 1953, is October 1st, 1953.

LONDON GRADUATES' AND STUDENTS' SECTION

A visit has been arranged to London Airport on Saturday, August 15th, and will include a tour of the various hangars, tunnels and runways in the course of erection. A coach will leave 11, Upper Belgrave Street, S.W.1, at 9 a.m., returning to Paddington at 1 p.m. The fare will be 6s. 6d., and as seats are limited, members wishing to take part in the visit are requested to communicate as soon as possible with the Honorary Secretary, who will book places upon receipt of the fare.

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Hon. Secretary : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES BRANCH—GRADUATES' AND STUDENTS' SECTION

Hon. Secretary : F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES BRANCH

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stonleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Hon. Secretary : S. G. Duckworth, M.I.Struct.E., " Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., " Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

Hon. Secretary : E. W. Howells, M.I.Struct.E., c/o Messrs. T. L. Harding & Sons, Ltd., 10-12, Market Street, Torquay, Devon.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone; 34-1111 Ext. 257.

Natal Section, Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

ADDITIONS TO THE LIBRARY

ABELES, P. W. *Principles and Practice of Prestressed Concrete*, Vol. 1, 2nd Edition, Revised. London, 1952.

American Society of Civil Engineers Manuals of Engineering Practice, No. 31. *Design of Cylindrical Concrete Shell Roofs.* New York, 1952. Presented by Dr. F. G. Thomas.

American S.T.M. 1952 Book of Standards, Pt. 1. *Ferrous Metals*, Pt. 2. *Non-Ferrous Metals*, Pt. 4. *Paint, Naval Stores, Wood, Fire Tests, Sandwich Constructions, Building Constructions.* Baltimore, U.S.A., 1952 and 1953.

Annual Report of the Smithsonian Institution, 1951. Washington, 1952.

British Constructional Steelwork Association Publication No. 6. *The Use of Welding in Steel Building Structures.* London, 1952. Presented by the Publishers.

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British Iron and Steel Institute Special Report No. 44. *Surface Defects in Ingots and their Products* (Recommended Definitions). London. 1951.

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DORMAN, LONG & Co. *Handbook for Constructional Engineers.* Middlesbrough, 1952.

FABER, OSCAR. *Simple Examples of Reinforced Concrete Design*, 4th Edition. London, 1952. Presented by Mr. C. H. Hockley.

F.B.I. *Register of British Manufacturers*, 1952-53, 25th Edition, London, 1952.

FLEMING BROS. (Structural Engineers), Ltd. *Pocket Section Book*, 8th Edition. Glasgow and London, 1952.

FRASER, J. M. *The Work of the Singapore Improvement Trust*, 1951. Singapore, 1952. Presented by the Publishers.

FULLER, W. H. *How to Draw Perspectives to Scale.* London, 1952. Presented by Mr. P. E. Sleight.

GOLDING, E. W., and GREEN, H. G. *Elementary Practical Mathematics*, Book I (First Year). London, 1947. 2nd Edition.

GUERRIN, A. and DANIEL, G. *Le calcul des tuyaux en beton arme et non arme.* Paris, 1952. Presented by Mr. P. J. Gerard.

HALASZ, R. VON. *Anschauliche Verfahren zur Berechnung von Durchlaufbalken und Rahmen.* Berlin, 1951. Presented by Dr. K. Hajnal-Konyi.

HALASZ, R. VON (Editor) *Holzbau-Taschenbuch*, 4th Edition. Berlin, 1952.

HUNTER, L. E. *Underpinning and Strengthening of Structures.* London, 1952. Presented by Mr. A. T. Wadi Williams.

Imperial Chemical Industries Codes and Regulations Group D, Vol. 1. 3. *Fire Prevention.* London, 1952. Presented by the Publishers.

Institute of Physics Memorandum on Gamma-Ray Sources of Radiography. London, 1952.

KLEINLOGEL, A. *Rigid Frame Formulas.* New York 1952. Presented by Professor J. A. L. Matheson.

MINIKIN, R. R. *Coast Erosion and Protection : Studies in Causes and Remedies.* London, 1952. Presented by Mr. K. F. Bird.

MOLLOY, E. (General Editor). *Architects', Builders' Civil and Highway Engineers' Reference Book.* London 1952.

MORGAN, T. LYLE. *Forces in Framed Structures* London, 1952. Two copies, presented by the Author and by Mr. J. McHardy Young.

MUSSON, W. A. J. and REID, R. D. *Exercises on Concise General Science.* London, 1952.

PEATFIELD, A. E. *Teach Yourself Mechanical Engineering*, Vol. II. *Engineering Components and Materials* and Vol. III *Workshop Practice.* London 1951 and 1952. Presented by the Author and by Mr. Newman Tate.

PEDOE, J. *Advanced National Certificate Mathematics* Vol. 1. London, 1951. Presented by Mr. E. J. Gouk.

PIPPARD, A. J. S. *Studies in Elastic Structures.* London 1952. Presented by Professor R. H. Evans.

REYNOLDS, C. E. *Examples of the Design of Reinforced Concrete Buildings in accordance with the B.S. Codes* London, 1952. Presented by Mr. W. Morgan.

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Cold Drawn Prestressing Wire

By J. L. Bannister, B.Sc.(Tech.), M.Sc., A.M.I.Struct.E.

Synopsis

n order to use available materials in the most efficient nner, it is essential for the user to have full knowledge the properties and characteristics of the material plied for prestressing tendons. The purpose of this per is to consider the various conditions in which cold wn high tensile steel wire can be supplied, and the ct of various treatments on the behaviour of this terial in terms of service conditions.

Previous work concerned with the behaviour of cold wn wire in various conditions is included, with estigations by the author on the stress-strain relation- p under increasing and decreasing tensile stresses, and relaxation of tensile stress from initial stresses ich are a relatively high proportion of the maximum ength. The effect of high temperature and tensile ess on the characteristics of wire in the as-drawn dition are considered, as well as the desirable charac- istics of prestressing tendons.

Introduction

The inherent weakness of concrete is its low tensile ength compared with its compressive strength. The vention of reinforced concrete is usually ascribed to nier, who in 1861 made several attempts to compen- te for this weakness of concrete in tension by reinforcing ncrete flower-pots with a mesh of iron wire.

Analytical treatment and later experiments were to ult in a better appreciation of the nature of reinforced ncrete, particularly its low tensile strain cracking pacity. According to Thomas, the first theoretical atment of prestressed concrete was due to J. Mandl in 96, followed by Koenen in 1907, the latter realising at a loss of the steel stress occurs due to the elastic rtening of the concrete. In 1908 C. R. Steiner ggested that stress losses due to shrinkage and elastic formation could be eliminated by successive stressing and when this became necessary.

The use of higher tensile steel stresses in the reinforce- ent resulted in an increase in the extent of cracking the concrete due to the greater strains, thus attention s directed towards attempts to eliminate the tensile akness of concrete by pretensioning the reinforcement, ich on release placed the surrounding concrete in mpression. In these early experiments mild steel s used as the prestressing tendon, and due to the k of knowledge of the shrinkage and creep movements the concrete, the resultant loss of steel stress was not preciated. These losses were of the same order as the mparatively small pretension force in the steel.

Freyssinet realised the necessity for the magnitude the preliminary tension in the steel to be large enough allow losses due to concrete movements, and still tain a substantial prestress in the steel.

Whilst the full use of high tensile steel as concrete nforcement would result in an increase of the extent cracking in the concrete, it is very efficient as a pre- ssing tendon. The development of the technique the production of high strength concrete and the use high tensile steel wire has resulted in the rapid velopment of prestressed concrete, and the charac- ristics of some of the high tensile steel wire used as pre-stressing tendons are examined in terms of service conditions.

Manufacture of Wire

Rods are manufactured from billets—approximately 2 in. square—which are first placed in a controlled temperature furnace where they are heated until in a suitable condition for reduction in size by hot rolling. The rods are continuously fed into sets of rolls where the cross section is appreciably reduced. As the rod passes through the set of rolls it is brought down to the desired size and automatically reeled into a coil in the hot condition.

During the rolling process the rod becomes coated with an iron oxide referred to as " mill scale," and it is essential to clean this scale off the rod before drawing it down into wire. This is carried out by dipping the rod coil into a dilute acid bath which removes the scale, and afterwards washing with hot water. The cleaned rod is then dipped into a lime solution to neutralise any traces of acid. When the lime has dried the rods possess a protective coating which also aids the wire drawing process.

To ensure uniformity of the material prior to cold drawing, the rod is heated to a sufficiently high temperature to promote a certain degree of grain growth, and cooled at such a rate as to yield a fine pearlite structure. During the cold drawing process the rod may be reduced in four to six drafts through a continuous wire drawing machine when the fine pearlitic equiaxial structure is broken down and drawn out into a fibred structure finely distributed.

Characteristics of Drawn Wire

The tensile test is the most extensively used mechanical test for assessing the suitability of a wire for use as a prestressing tendon. From the stress strain curve, the value of a stress corresponding with a certain inelastic elongation, and known as a proof stress, along with the maximum stress is determined. The determination of these two stresses may serve as a suitable acceptance test, and indicates that the wire has a certain strength, but it gives little indication of the behaviour of the wire under prestressing service conditions. The shape of the stress-strain curve is of great importance in assessing the suitability of wire for use as a prestressing tendon, as this is dependent on the complex behaviour of drawn wire which in turn is dependent on the shape of the specimen, the rate of straining, the previous strain history and the non-elastic characteristics of the wire.

A typical nominal stress-strain curve for a patented and drawn wire is shown at (1) in Fig. 1, which displays the absence of a limit of proportionality, a yield point, and any elastic characteristics. If the wire (1) is tensioned to a load which is about 70 per cent. of the maximum, and then unloaded to zero, the deformation is not reversible, and the wire acquires a permanent deformation of 0.14 per cent. This deformation is, however, recoverable after a period of time, and when the wire is reloaded the stress-strain curve is of the same form as the original. If, immediately after returning the load to zero, the wire is reloaded again, the strains

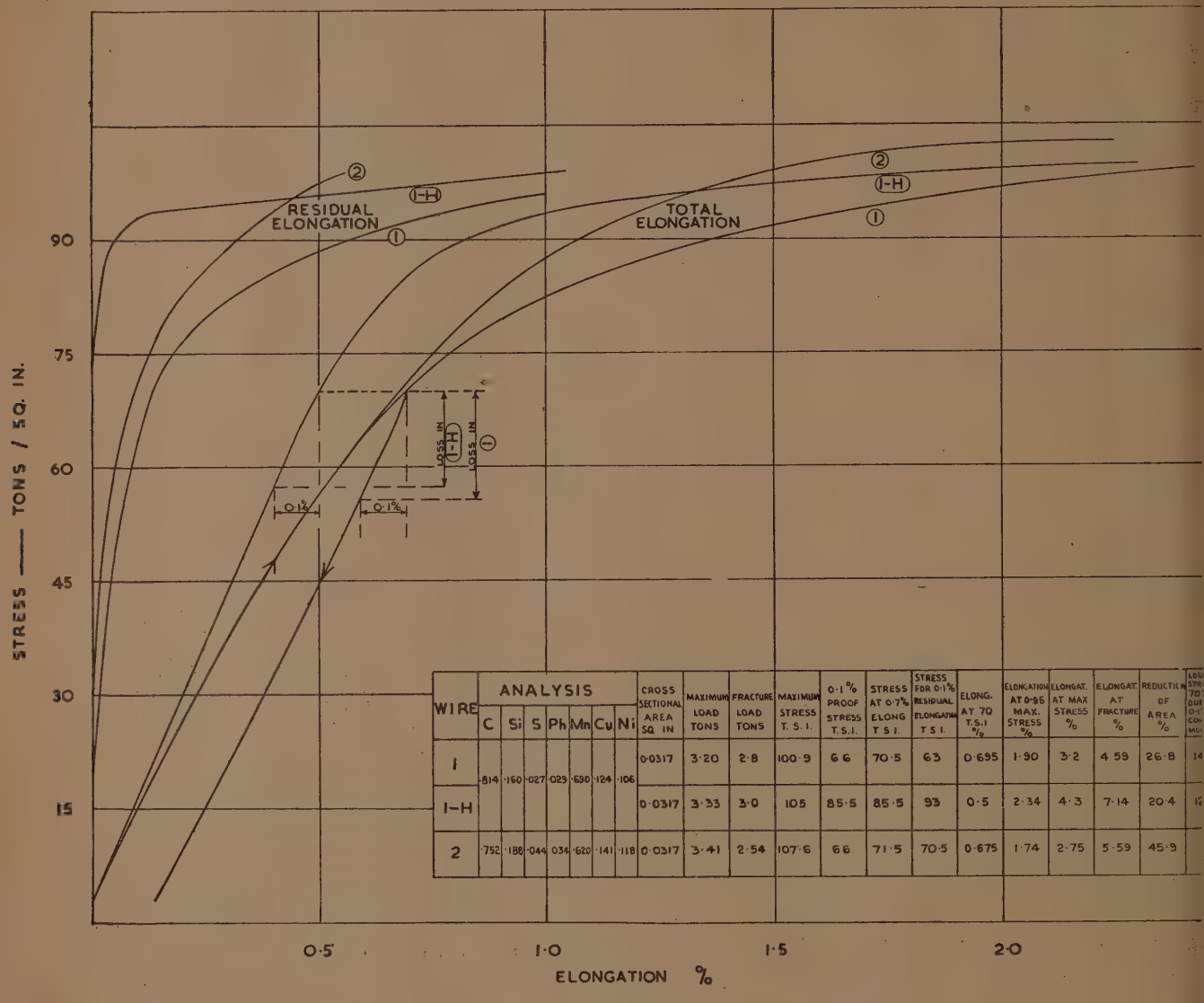
do not coincide with those of the first load removal but form a loop as shown in Fig. 2.

The effect of previous strain history on the shape of the stress-strain curve has been shown by Brown, who subjected a patented and drawn wire to 20 repetitions of a tensile load equal to 75 per cent. of the maximum. After this treatment the wire tends to display less plastic behaviour, but if the wire is now subjected to 50 reversals

is clearly shown in the residual elongation curves for t two wires in Fig. 1.

Stress-Strain Relationship

The results of the tensile test are expressed as nominal stress-strain curve, and it is from the shape this curve that the suitability of a wire for use as a pr stressing tension is judged. If a wire with an origin



WIRE	ANALYSIS C	Si	S	Ph	Mn	Cu	Ni	CROSS SECTIONAL AREA SQ IN	MAXIMUM LOAD TONS	FRACTURE LOAD TONS	MAXIMUM STRESS T. S. I.	0·1% PROOF STRESS T.S.I.	STRESS AT 0·7% ELONG T S I.	STRESS FOR 0·1% RESIDUAL ELONGATN T S I.	ELONG. AT 70 T.S.I %	ELONGATION AT 0·95 MAX. STRESS %	ELONGAT. AT MAX STRESS %	ELONGAT. AT FRACTURE %	REDUCTION OF AREA %
1	·814	·160	·027	·023	·690	·124	·106	0·0317	3·20	2·8	100·9	66	70·5	63	0·695	1·90	3·2	4·59	26·8
1-H								0·0317	3·33	3·0	105	85·5	85·5	93	0·5	2·34	4·3	7·14	20·4
2	·752	·188	·044	·034	·620	·141	·118	0·0317	3·41	2·54	107·6	66	71·5	70·5	0·675	1·74	2·75	5·59	45·9

Fig. 1

over a pulley whose diameter is 115 × the wire diameter, then the original plastic behaviour is restored.

All patented and drawn wire displays a non-elastic stress-strain curve and the type of deformation is dependent on both applied stress and time. This plastic behaviour means that the wire can be subjected to more severe deformations without reaching the maximum load, and thus a greater degree of warning of failure is given in the case of over-stressing.

The residual elongation curves or flow curves in Fig. 1 represent the permanent part of the total elongation at zero stress after unloading from a particular stress. Wire (2) was produced by smaller reductions of area in the drawing process than wire (1), the former displaying less plastic characteristics than the latter. This difference

volume $A_0\, l_0$ is extended until the length is l and t reduced cross sectional area is A, then the strain

$$e = \frac{l_0 - l}{l_0} = \frac{A_0}{A} - 1 \quad . \quad . \quad . \quad . \quad . \quad . \quad .$$

whereas the true strain ε is the summation of the chan in length divided by the average length during the chan in length, and provided the volume remains consta

$$\varepsilon = \log \frac{A_0}{A} = \log (1+e) \quad . \quad . \quad . \quad . \quad .$$

According to Kuntze and Sachs the non-uniformity deformation proceeds in a series of local extensio

companied by a corresponding local contraction. At is point of local contraction an increase in stress occurs id some strain-hardening. If the rate of strain-ardening is greater than the rate of reduction of area, ien the load carrying capacity of the wire increases and ie local plastic deformation is transferred to another ss strain-hardened section. As soon as the strain-ardening fails to compensate for the reduction of area, ecking sets in and the maximum load has been reached.

tion proceeds beyond the strain-hardening resistance. Hence the maximum load is slightly above the necking load.

The condition for maximum load from equations (3), (1) and (2) is given by

$$\frac{df}{f} = \frac{de}{1+e} = d\epsilon \quad \ldots\ldots\ldots \quad (4)$$

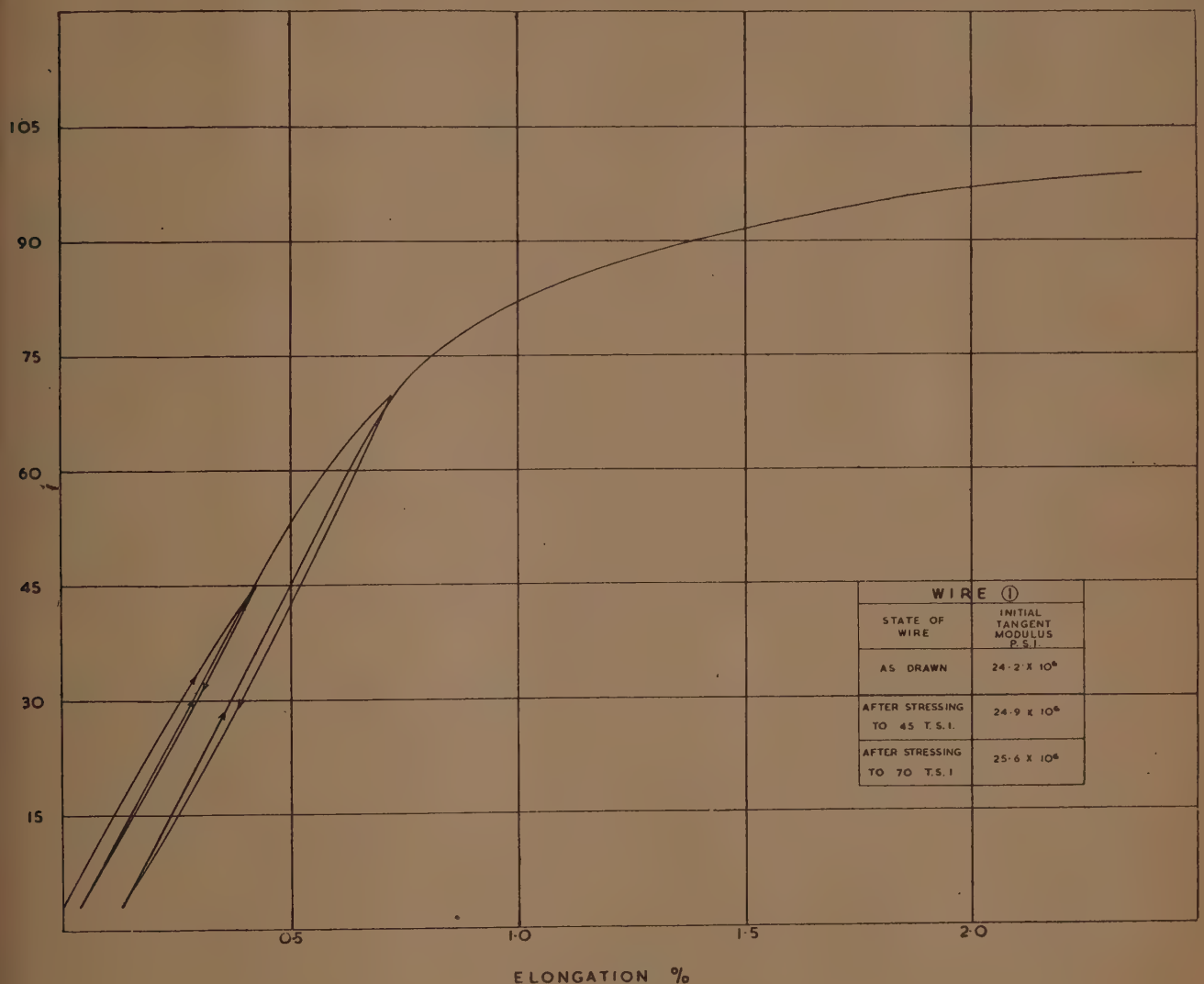


Fig. 2

he true stress up to the point of necking is the intensity f the tensile force on the instantaneous area.

Maximum load $P = f.\ A$

$$dP = A.\ df + f.\ dA = 0$$

$$\frac{df}{f} = -\frac{dA}{A} \quad \ldots\ldots\ldots \quad (3)$$

hus at the maximum load the increase in the total orce $(A.\ df)$ = the decrease due to the reduction of rea $(-\ f.\ dA)$; the occurrence of maximum load ndicating a change in the strain-hardening of the wire nly and not an ultimate strength.

Freudenthal suggests that necking starts just before he maximum load is reached, because the resistance to eformation is a function of the rate of straining as well s strain-hardening. Necking tends to increase the ate of straining, and raises the stress at which deforma-

which means that the maximum load occurs at a strain at which the slope of the true stress-strain curve becomes equal to the stress. The maximum load for all wires is recorded in Figs. 1 and 11, and the maximum stress is calculated from the original area. The original area is 1.0329 of the instantaneous area at the maximum load in the case of wire (1) which means that the true maximum stress is more than 3 per cent. greater than the nominal maximum stress, while in the case of wire (2) the true maximum stress is over $2\frac{1}{2}$ per cent. greater than the nominal.

The 0.1 per cent. proof stress is determined for drawn wire by drawing a line parallel to the initial tangent modulus through 0.1 per cent. elongation. The slope of the tangent depends on the precision with which the deformations are measured and the rate of loading. Gurney presents evidence to show that an increase in the rate of loading results in both a reduction in the

amount of deformation associated with a given stress, and a small increase in the maximum stress. Thus the curvature of the stress-strain curve is subject to variation and in turn the slope of the tangent and the value of the proof stress.

Furthermore the proof stress does not give any indication of the behaviour of the wire as a prestressing tendon ; a more suitable indicator being the stress at an

deformation which takes place up to the point of necking and the other a non-uniform deformation confined t the region of necking.

The meaning of ductility from the engineering poin of view is the amount of deformation that the materia can withstand before failure. If the uniform deforma tion is exceeded, the instability condition of neckin starts in a local region and fracture follows under an

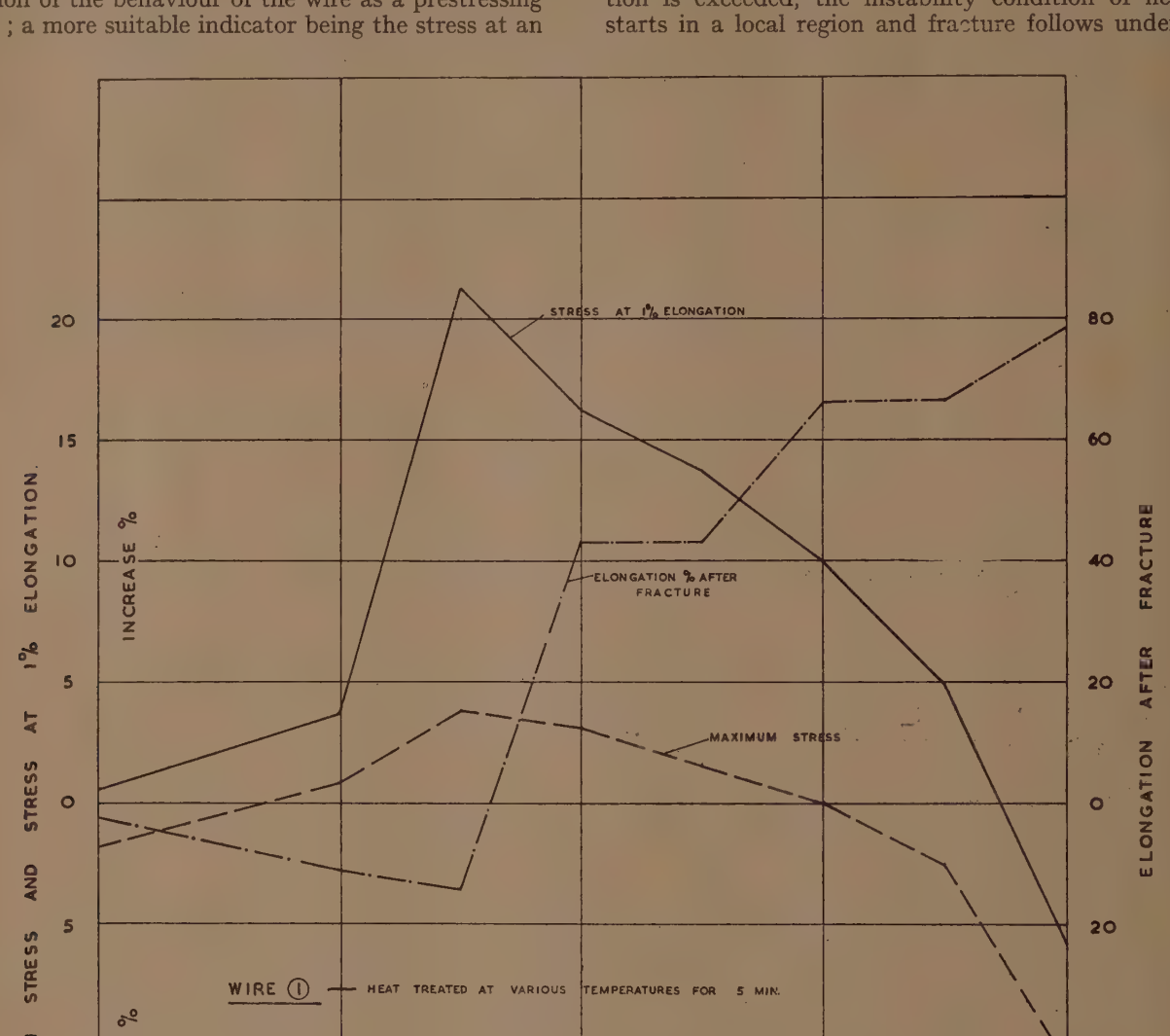


Fig. 3

elongation of say 0.7 per cent. As previously stated, the stress-strain characteristics of the wire are affected by previous strain history, less plastic behaviour being apparent by a decrease in the curvature of the stress-strain curve after a loading and unloading cycle. The effect of loading and unloading cycles on the proof stress are shown in Fig. 2.

The ductility of wire is usually expressed in terms of the total elongation after fracture has taken place. This elongation is composed of two parts, one a uniform

further deformation. Thus the uniform deformatic is related to service conditions and indicates

(i) the limit of the deformation which can be applie in service before the wire fails,

(ii) the capacity of the wire to absorb energy und excess loads.

Since the uniform deformation ends at or about tl maximum load then the total elongation at this loa gives the maximum elongation possible before failur

lloman gives the relation between stress and strain to the maximum load as

$$f = k.\ \varepsilon^n \quad \ldots \quad (5)$$

ere n is the strain-hardening exponent.
om equation (5)

$$\frac{df}{f} = n.\frac{d\varepsilon}{\varepsilon} \quad \ldots \quad (6)$$

The proportion of the total elongation of the wires at fracture that takes place as a non-uniform elongation in the form of necking is proportional to the reduction of area. This latter quantity is thus only a measure of the deformation in the range of necking.

The conditions at fracture consist of triaxial stress which commences and increases with necking, and a fracture stress which is difficult to interpret unless the necking is small and brittle fracture ensues. These conditions are not related to service conditions.

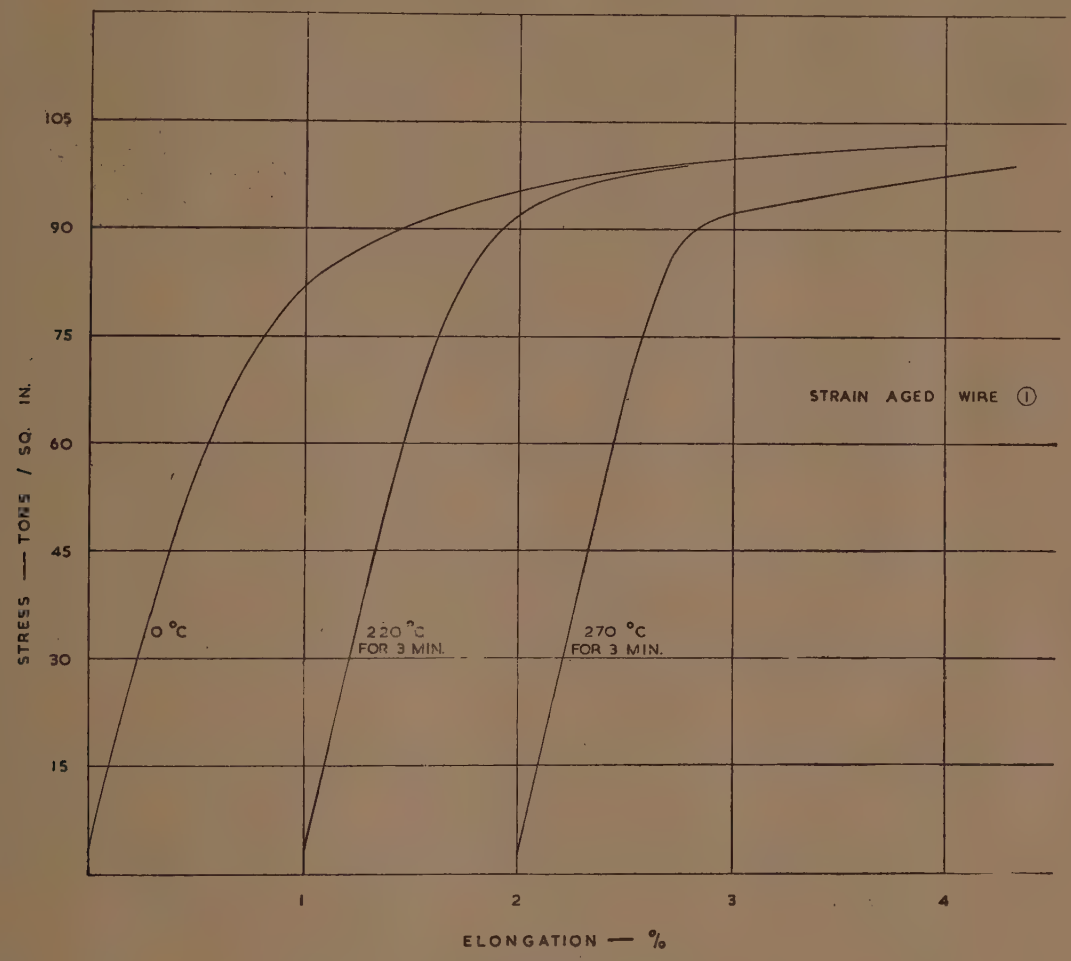


Fig. 4

id from equations (4) and (6) it follows that the uniform rain ε_m at maximum load is equal to the strain-ardening exponent n.

By careful manipulation of the testing machine the aximum load can be held long enough to read the tal elongation corresponding to this load, and to move the extensometer prior to necking and subse-ient fracture. Also true strain at the maximum load n be determined by measuring the reduced diameter the wire at this load and using equation (2). These eformations are shown in Table 1.

When measuring the original diameter the wire should e rotated so as to disclose any ovality of the cross ction which is usually of the order of $\frac{1}{2}$ per cent. It is r this reason that the determination of the uniform ongation from the reduced diameter is subject to some ariation. Once necking has started, the wire continues deform in this region, while away from this region the niform deformation ceases. So that in a specimen, y 80 to 100 diameters long, the reduced diameter rresponding to the maximum load can as an alter-ative be measured immediately after fracture.

TABLE 1

Wire	Elongation—% at Maximum Stress from			Elongation—% at Fracture	Reduction of Area %
	Extensometer	Reduced Diameter			
		Equation (1)	Equation (2)		
1	3.20	3.29	3.24	4.59	26.8
2	2.75	2.56	2.53	5.59	45.9
1-H	4.30	4.50	4.39	7.14	20.4
1-H-T	4.20	4.12	4.02	6.1	20.0

Strain-Aged Wire

Strain-ageing is a term used to describe the alteration in the characteristics of a wire after previous cold

deformation. This strain-ageing can take place at ordinary temperatures, but the rate of change increases with temperature. Strain-ageing of the wire will produce an alteration in the shape of the stress-strain curve which makes it possible to realise more desirable characteristics.

The application of a controlled temperature for a specific time will result in more elastic characteristics, or the application of a tension will subsequently improve the elastic characteristics up to this tension. Brown shows that when a wire is tensioned to ½ maximum stress 20 x, the elastic characteristics are improved but not completely restored. If the wire is heated to 210°C. for 20 minutes and then tensioned to ½ maximum stress 20 x the elastic properties are enhanced still more. Brown also maintains that if a wire is tensioned prior to heat treatment then completely stable elastic characteristics are obtained up to that tension, and the width of the loading and unloading loop is considerably reduced.

Strain-ageing by heat-treatment was applied t Wire (1) by immersing in a lead alloy bath for a period c five minutes. The temperature of the bath was care fully maintained at various temperatures betwee 100°C. and 500°C. By plotting the variation in tensil strength against temperature in Fig. 3, it can be see that the strength first falls and then rises to a maximum falling again at higher temperatures. The effect of th

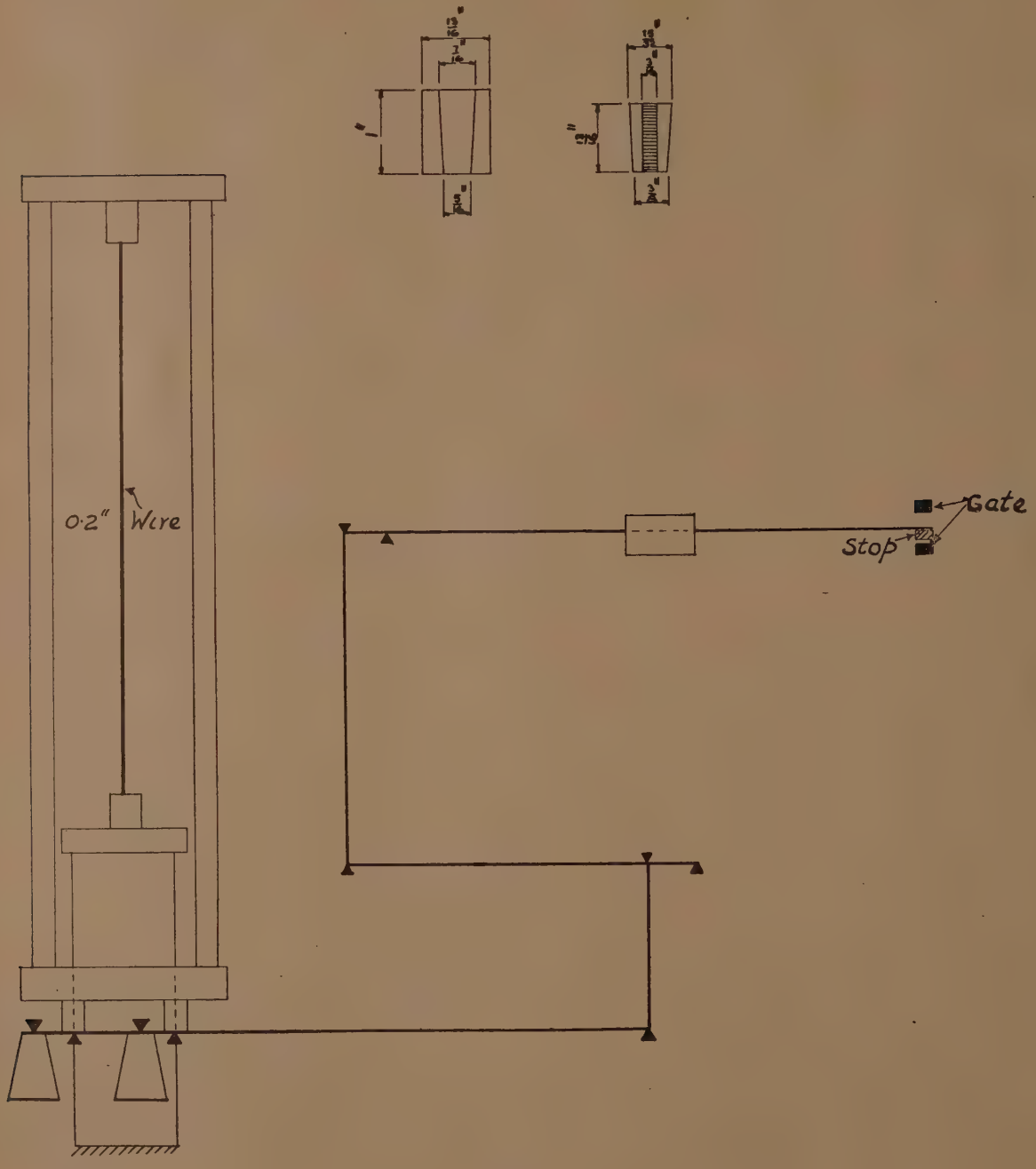


Fig. 5

heat-treatment on the stress-strain characteristics i shown in Fig. 4, where the stress-strain curves ar plotted for the Wire (1) before and after being submitte to temperatures of 220°C. and 270°C. for a period c three minutes.

The stress-strain curve for Wire (1) before and afte heat-treatment (1-H) is shown in Fig. 1. The latte displays more elastic characteristics than the former The ductility of the strain-aged wire is also improved with a greater degree of warning of failure due to th increase in elongation between 70 t.s.i. and the maximun

ess. The unloading curve from a stress of 70 t.s.i. is very little different from the loading curve for Wire (H) that it is difficult to plot, while the unloading ve for the Wire (1) in the as-drawn condition is shown form a loop with the loading curve. For the same ncrete deformation due to prestress, the loss of stress in case of (1-H) is smaller than in the case of Wire (1).

(a) Concrete deformations,
(b) Relaxation of stress in the steel,
(c) Movements in the anchorage.

The first item consists of compressive, shrinkage and creep movements and for a post-tensioned concrete subjected to a prestress of 1500 p.s.i. with a secant modulus of 5×10^6 p.s.i., the values of the shrinkage

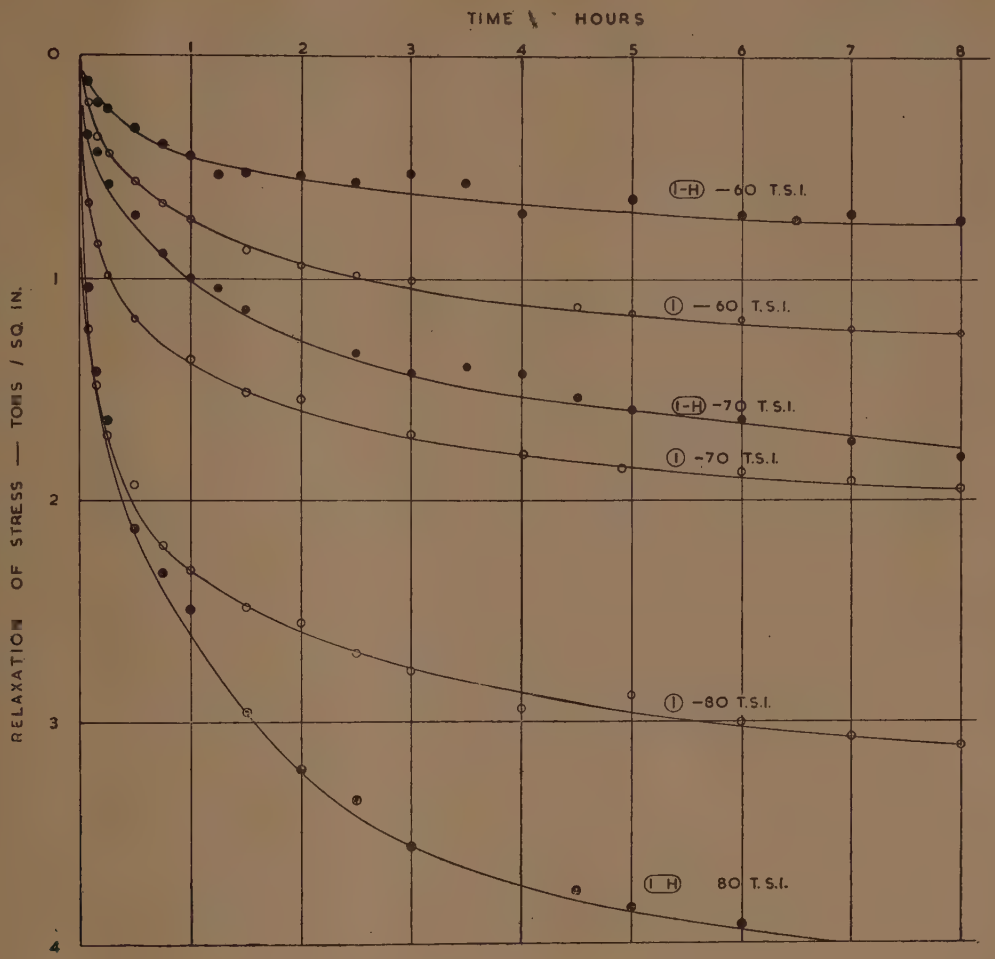


Fig. 6

It will also be seen from Fig. 1 that the strain-aged ire has no residual elongation until it has been stressed about 75 t.s.i.

Relaxation of Stress

In ordinary reinforced concrete construction the eel stress is proportional to the loads applied to the eam, while in prestressed concrete construction the nount of loss of steel stress by concrete deformation greater than the increase due to the application of the sign load. Thus the preliminary prestress is the eatest tension applied to the wire and since the value this stress can be readily measured it can with safety a high proportion of the maximum stress.

The First Report on Prestressed Concrete published the Institution of Structural Engineers (7.(b) p. 11) lows a preliminary stress of 60 per cent. to 70 per cent. the maximum tensile strength of the wire.

If the strain corresponding to the high preliminary ress is held constant, then at ordinary temperatures, laxation of the preliminary stress takes place with me. This is known as " Simple Relaxation."

The loss of tensile stress from the preliminary tension the prestressing wire is due to the following causes :—

and creep movements are 0.02 per cent. and 0.03%/1000 p.s.i. respectively. Hence the loss of preliminary tension due to the deformations which constitute (a) correspond to a deformation of 0.065 per cent. For the Wire (1) in the as-drawn condition the loss of stress from a preliminary tension of 70 t.s.i. is about 10 t.s.i., while for Wire (1-H) the loss of stress from the same preliminary stress of 70 t.s.i. is about 8 t.s.i. due to this concrete deformation of 0.065 per cent.

The loss of stress due to relaxation of tension after 10 days from a stress of 60 t.s.i., is about 2 t.s.i. for Wire (1) which is approximately the same as the loss due to shrinkage movement of the concrete.

The loss of stress due to movements in the anchorage can be appreciable. A slip of 1/16 in. on a length of 15 ft. is equivalent to a movement of approximately 0.035 per cent., which corresponds to a loss of stress from 70 t.s.i. of about 5 t.s.i.

Mechanism of Relaxation of Stress

In a strain-hardened material such as drawn wire, the deformation is nearly all concentrated in slip planes which are usually those in which the densest packing of atoms occurs. The slip which takes place results in a

resistance to further deformation along the same plane. This resistance to deformation is attributed to the difficulty of moving a dislocation, i.e., a region of imperfection in the crystal lattice.

According to Taylor, a dislocation will move when the applied stress is equal to the maximum internal stress. When the internal stress is high, the slip spreads on a number of sets of planes, and the dislocations do not travel right across a slip plane, but become jammed by the interlocking of one with another. An intense centre of strain is formed which prevents the reversibility of plastic flow and causes strain-hardening.

Cottrell and Bilby interpret strain-hardening in terms of the migration of carbon atoms to free dislocations. It is suggested that carbon atoms form a cloud around a dislocation, and a migrating movement is trying to move the atom cloud, while it resists.

Mott maintains that the resistance to deformation is due mostly to the difficulty of moving dislocations, and with increasing strain the number of dislocations increases. On application of the initial stress, the initial instantaneous relaxation of stress occurs by " exhaustion " of dislocations, and the value of internal stress around all dislocation loops is now greater than the external stress, and further relaxation takes place by the aid of the thermal stress fluctuations.

Factors Affecting Relaxation

In general a fine grained material is more creep resistant than a coarse grained material at ordinary temperature.

The effect of heat-treatment on relaxation or cre characteristics is indirect, and depends on the effect the heat-treatment on such factors as grain size a structural stability.

The combined effect of heat treatment and tensi on the relaxation or creep characteristics is also indire and depends on the treatment being applied long enou for a condition of structural equilibrium to be obtain

Previous Work on Relaxation

The variable characteristics of wire in the as-dra condition, although manufactured by the same proce was shown to apply to the relaxation of stress. Strycker found the variation in relaxation losses f wire from the same batch to be as follows in Table 2.

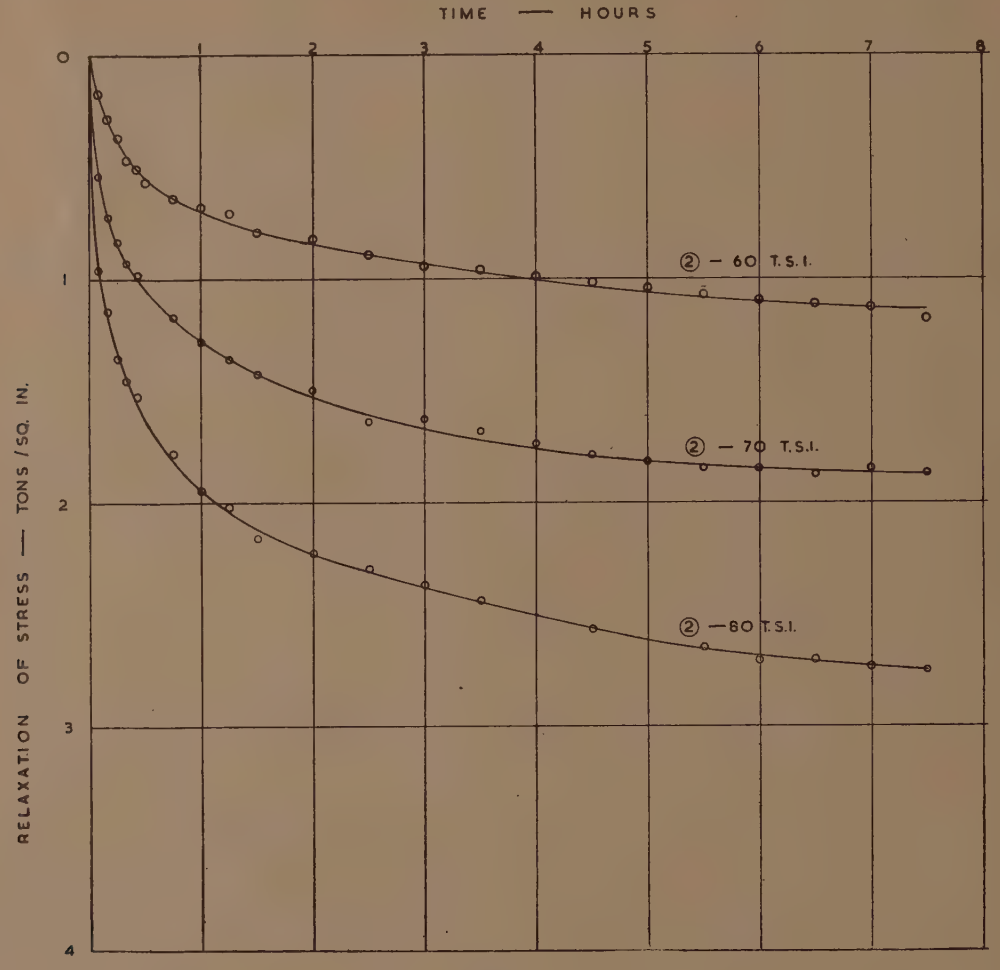


Fig. 7

TABLE 2

	Relaxation of Stress %	After a Period hours
1st Case ...	2	1
	4	24
2nd Case ...	1.65	1
	3.50	24

de Strycker also observed that a naturally strain-age wire has a greater relaxation loss than a wire with small age. He found that a new wire relaxes 3 per cen in three days from an initial stress of 130 kg/sq. mm while the same wire when two years old would the relax the same proportion from an initial stress of on 110 kg/sq. mm. Everling also showed that for

08 in. dia. wire when subjected to an initial stress ual to 70 per cent. of the maximum stress, the relaxa- on at the end of 120 hours was 44 per cent. greater for e wire in the artificially aged condition.

The beneficial effect on the relaxation of wire of a mporary overstress has been demonstrated several nes.

de Strycker showed that if the wire is stressed up to kg/sq. mm. for two minutes and then reduced to kg/sq. mm., the creep is only 5/7 of the value obtained 85 kg/sq. mm. Schwier reduced the creep by stressing to the 0.2 per cent. proof stress for five minutes. verling tensioned a wire to 80 per cent. of the maximum ress for two minutes, and then reduced it to 70 per cent.

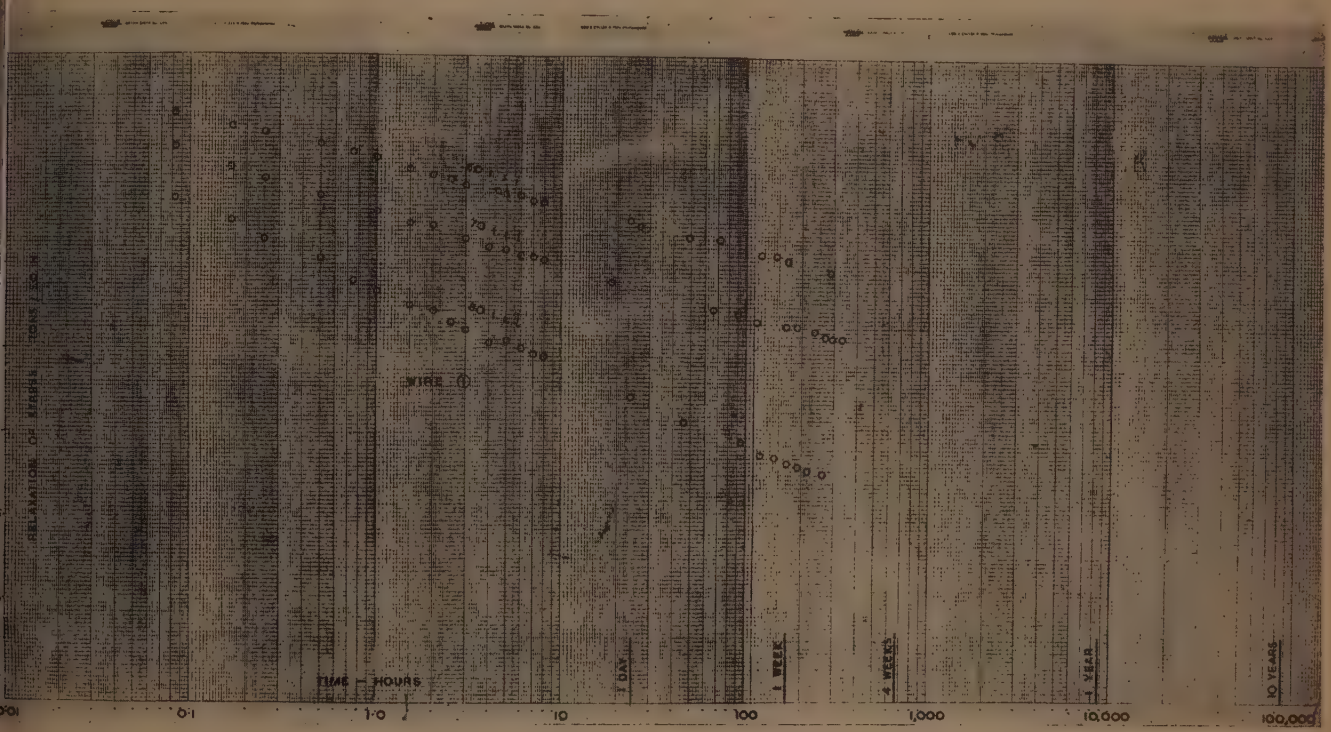


Fig. 8

he relaxation from this initial stress was 56 per cent. ss than the relaxation of the original wire from the me initial stress at the end of 120 hours.

Magnel testing 5 mm. diameter wire showed that by ressing to 137,000 lb./sq. in. for two minutes and then educing the stress to 123,000 lb./sq. in. the relaxation om the latter stress after 300 hours was reduced 7 per cent.

Method of Testing

Simple relaxation tests were carried out on the wires presented in Fig. 1 from initial stresses of 60, 70 and tons/sq. in. While these initial stresses are of the der of those used in service, it is to be noted that the rain does not remain constant in service due to the ncrete deformations that take place under load. hese may be of the order of 10 per cent. to 20 per cent. the steel strain.

A lever machine using dead weights is the most nvenient method of carrying out a relaxation test. he initial load is applied and the wire is allowed to ongate. At the end of a short time, further elongation prevented by stopping any downward movement of the beam. From this time onward the wire is held at a constant strain by having sufficient load on the beam to hold it against a stop. After a short period of time the wire has relaxed and it is found that the load required to hold the beam on to the stop can be reduced.

The machine used for the relaxation tests was a compound lever type shown in Fig. 5 which recorded the load direct to 1 lb. A 36 in. length of wire was used for the test. Split conical wedge grips shown in Fig. 5 were forced on to the ends of the wire in a vice 24 hours prior to testing, and the whole allowed to remain unloaded in the testing machine so that all equipment is at approximately the same temperature. Immediately prior to the start of the test the wedges are again subjected to a force, so that the test is started with the grips tight on the wire.

The load corresponding to the initial stress is applied in less than one minute, and the relative movement between the wire and the grips is measured on a dial gauge. This load is maintained for one minute, and if at the end of this time the relative movement of wire and grip has not ended, the test is abandoned and a new specimen is prepared.

During this initial period of one minute the weigh lever arm is maintained at the mid position of a gate ; at the end of this period a stop is placed under the lever arm. The elongation of the wire has been stopped and the wire now relaxes its load. This relaxation is measured by reducing the load at the end of a further five minutes until a film of light is visible between the bottom of the beam and the top of the stop. The presence of this film is sensitive to 1 lb. of load. During the first eight hours frequent load readings are taken, when the temperature is steady, the specimen being shielded so as to avoid the temperature changes in the specimen being more rapid than in the machine. After the first day, load readings are taken daily when the

temperature is steady, the most suitable time being at the beginning of the morning.

Relaxation of Drawn Wire

The relaxation of stress with time up to eight hours for Wires (1) and (2) are shown in Figs. 6 and 7, where it can be seen that the immediate rate of relaxation of stress is rapid, but due to the reduction of load and the strain-hardening during test, the rate of relaxation decreases with time.

After a short transition a more steady rate of relaxation is observed and according to Bailey it is due to the rate of temperature softening compensating the rate of strain-hardening. Clarke and White attribute this compensation to recrystallisation. It is probable that this compensation can also be attributed to other time-temperature phenomena.

The higher the initial stress, the greater the relaxation, but the residual stress is also higher. The relaxations from initial stresses of 60, 70 and 80 t.s.i. against log-time, for periods of 250 hours, are shown in Figs. 8, 9 and 10.

The relaxation of stress which takes place in the first hour forms a substantial portion of the relaxation at 10 days. The ratio of these two values increases with the initial stress. In the relaxation tests carried out by Dawrance the stress in the 5 mm. dia. wire was determined by measuring its natural frequency of transverse vibration. A comparison of the results of relaxation obtained by the two totally different methods is of interest, and is shown in Table 3.

TABLE 3

Initial Stress t.s.i.	Relaxation of Stress at 10 days t.s.i.			Relaxation of Stress at 1 hour / Relaxation of Stress at 10 days %		
	5 mm. Wire	Wire (1)	Wire (2)	5 mm. Wire	Wire (1)	Wire (2)
60	2.09	2.08	1.55	36.4	35.2	27.2
70	2.86	2.80	2.75	33.3	48.7	46.7
80	4.19	4.48	4.16	33.3	51.6	46.7

The difference in the relaxation losses of Wire (1) and (2) shows the possible variation which can occur in the characteristics of wires in the as drawn condition, although the process of manufacture and diameter are the same. These variable characteristics were also demonstrated by de Strycker.

Fig. 9

Relaxation of Strain-Aged Wire

The relaxation of stress for periods up to eight hours for Wire (1-H) is also shown in Fig. 7. This is the Wire (1) in the artificially aged condition. For periods up to 250 hours the relaxation for Wire (1-H) is shown in Fig. 10.

It was found that due to artificial strain-ageing the relaxation from an initial stress of 60 t.s.i. is reduced while from an initial stress of 70 t.s.i. the relaxation of the strain-aged wire approaches that of original drawn

e after about 10 days. If the initial stress is 80 t.s.i. s clear from Fig. 6 that the relaxation of stress for ain-aged wire is substantially greater than that for same wire in the as-drawn condition.

t appears, therefore, that the relaxation of stress of a ain-aged wire is less than that of the same wire in the drawn condition if the initial stress is not greater n 60 t.s.i.

laxation of Wire Strain-aged while in Tension

The Wire (1) was subjected to the artificial strain-ing process while under a tension of 74 t.s.i. The ess-strain curve after this treatment is shown in Fig.

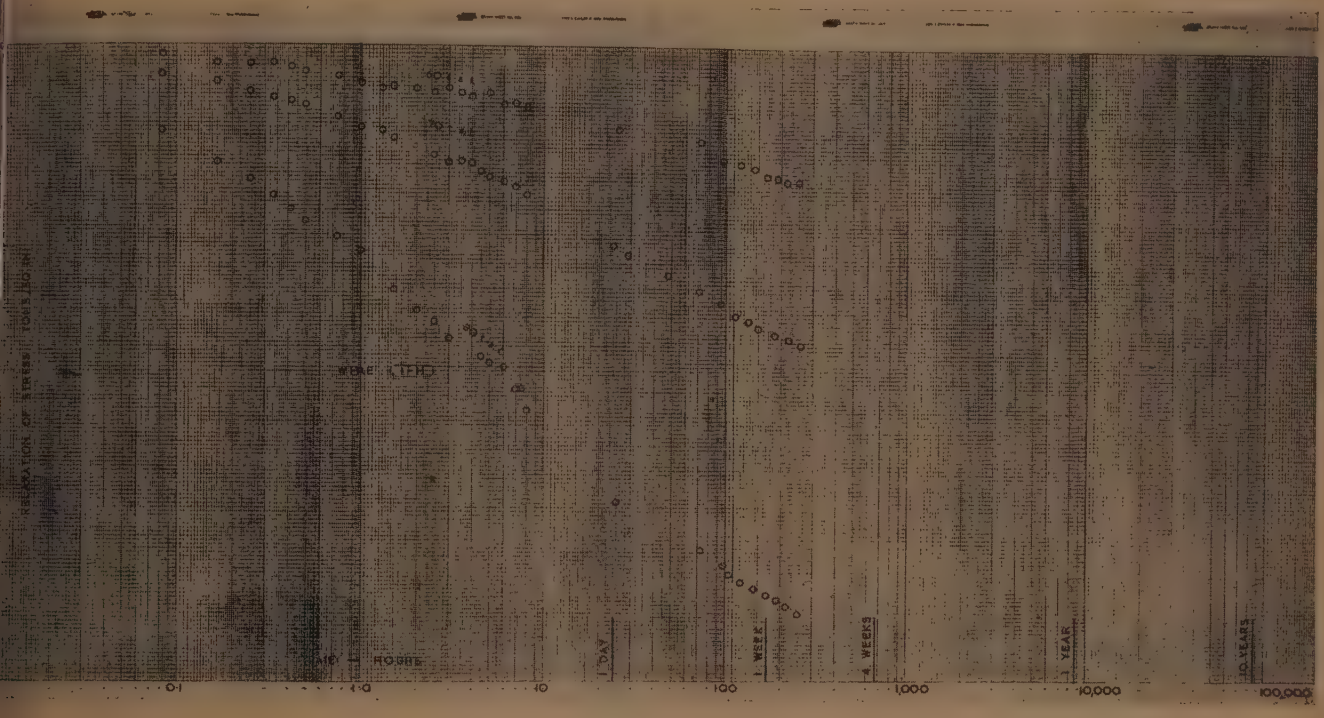


Fig. 10

. This may be described as strain-ageing and strain-rdening the wire simultaneously.

The effect of this treatment is shown in Fig. 12 where e relaxation of stress for the wire (1-H-T) from initial resses of 70 t.s.i. and 80 t.s.i. is less than that for ire (1) from the same initial stresses.

Similar results have been obtained by Schwier; these e shown in Table 4, with results determined by the thor for Wire (1-H-T).

TABLE 4

Wire	Initial Stress, t.s.i.	Diam. in.	Relaxation after 100 hours, t.s.i.	
			As Drawn	Heat-treated under tension
thor ...	70	0.2	2.63	0.598
hwier ...	73	0.197	2.54	0.635

Strain-Hardened Wire

An example of strain-hardening is shown in Fig. 13. a given stress the elongation % is greater for the 0.2 in. dia. crimped wire than for the 0.2 in. dia. plain wire. After the application of an initial stress of 60 t.s.i. for 10 days, the crimped wire was left unloaded for three days. The elongation % at a given stress is now smaller, and the stress-strain curve for the hardened crimped wire is very similar to that for the plain wire, although he crimps are still present.

Effect of Temperature on Wire (1)

When the unstressed Wire (1) has been heated to various temperatures and allowed to cool, it will be seen in Fig. 3 that the tensile strength is either unaltered or increased if the temperature lies in the range 200°C. to 400°C. On the other hand, if the wire has been heated to a temperature outside this range then the tensile strength is reduced.

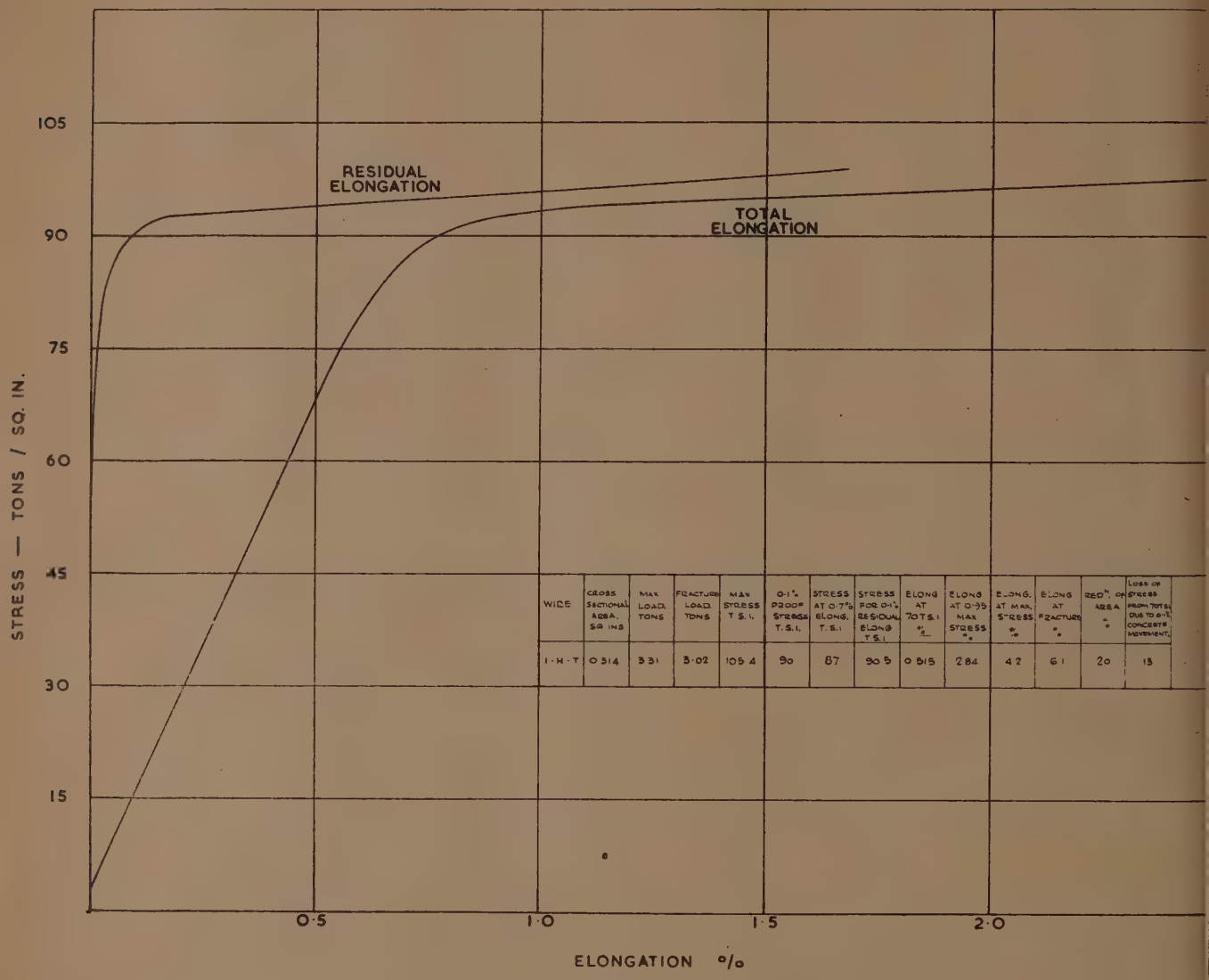
According to the Library Abstract No. 1, published by the Cement and Concrete Association—translated from the Dutch—it was found in high temperature tests on a prestressed concrete beam, that while the concrete reached a temperature of over 700°C., the temperature of the wire rose to only 230°C.—270°C. Samples of the wire were taken from the fire tested beam, and the tensile test showed that the tensile strength was unaltered. It is likely that the range of temperature to which the Dutch wire must be previously heated, in order that the tensile strength shall remain unaltered, includes the temperatures 230°C.—270°C.

The most important effect of temperature on pre-stressing wire is not the subsequent effect on the tensile strength after cooling, but the effect on the strain in the wire, which is subjected to temperature and stress simultaneously. To investigate the effect of temperature on the strain of a highly stressed wire a length of 0.2 in. diameter wire 10 ft.—9 in. long was tensioned in a prestressing bed by means of a hydraulic jack. The load on the wire was measured by a proving ring, and it was heated by passing an electric current along its length. A portion of the wire was polished, and the

temperature was judged by the appearance of the following characteristic colours :

Dark Straw 240°C.
Dark Blue 300°C.

The wire was first loaded up to the initial tension (50, 60 or 70 t.s.i.) in equal increments of 3 t.s.i. After each load increment the extension of the wire on a 2 in. gauge length, and the total elongation on the length of 129 in. was measured. The latter quantity being corrected for movement of the wire in the grips. At 60 t.s.i. the extension due to movement of the stressed wire in the grips was over 10 per cent. of the total extension. The stress-strain relations for the two gauge lengths are the same. The current was passed through the wire until the polished portion acquired the characteristic colour corresponding to either 240°C. or 300°C. As the wire elongated due to the rise in temperature, the load on the wire was maintained at a constant. As the wire reached the desired temperature the current was immediately switched off and the maximum elongation of the wire under the combined effects of stress and temperature was measured. As the wire contracted, the load was again maintained at a constant, and on reaching the normal temperature the elongation under the initial stress was found to be greater than before heating took place. After such treatment one would expect th characteristics of the wire to have been altered. Thi was found to be the case when the wire was reheated ; th elongation due to heating was deduced. The results ar shown in Fig. 14 and Table 5, and are applicable to post tensioned concrete.



WIRE	GROSS SECTIONAL AREA SQ INS	MAX LOAD TONS	FRACTURE LOAD TONS	MAX STRESS T.S.I.	0·1% PROOF STRESS T.S.I.	STRESS AT 0·7% ELONG. T.S.I.	STRESS FOR 0·1% RESIDUAL ELONG T.S.I.	ELONG AT 70 T.S.I °/o	ELONG AT 0·99 MAX STRESS °/o	ELONG. AT MAX. STRESS °/o	ELONG AT FRACTURE °/o	RED^N OF AREA °/o	LOSS OF STRESS FROM 70 T.S.I. DUE TO 0·1% CONCRETE MOVEMENT.
I·H·T	0·514	3·31	3·02	105·4	90	87	90·5	0·515	2·84	4·2	6·1	2·0	1·3

Fig. 11

Table 5

Tensile Stress t.s.i.	Strain % after				
	Stress	Temperature of 240°C.	Cooling	Temperature of 300°C.	Cooling
2	0.013	0.34	0	0.48	0
50	0.425	0.70	0.41	1.19	0.87
60	0.55	1.27	0.97	2.71	2.47
70	0.69	3.03	2.56	—	—

The stress-strain relationship at a temperature o 240°C. is given by the results in Table 5, where it will b noted that the extension of the wire is considerabl increased by the application of stress and temperature This will account for the conclusion in the Cement an

ncrete Association Library Abstract No. 1 that The amount of deformation is to a great extent verned by the load on the beam."

After cooling there is a considerable amount of sidual strain. This coincides with the conclusion ached by the Fire Research Station—" Beams which ve been exposed to a fire of shorter duration than that hich would cause failure representing say, less than

where w is the uniform load necessary to maintain equilibrium.

From equations (7) and (8)

$$e^2 = (s-l)\tfrac{3}{8}l \quad \therefore e = f_1(s, l) \quad \ldots\ldots\ldots (9)$$

and $$H = \frac{wl^2}{\sqrt{(s-l)\ 24l}} \quad \therefore H = f_2(s, l) \quad \ldots (10)$$

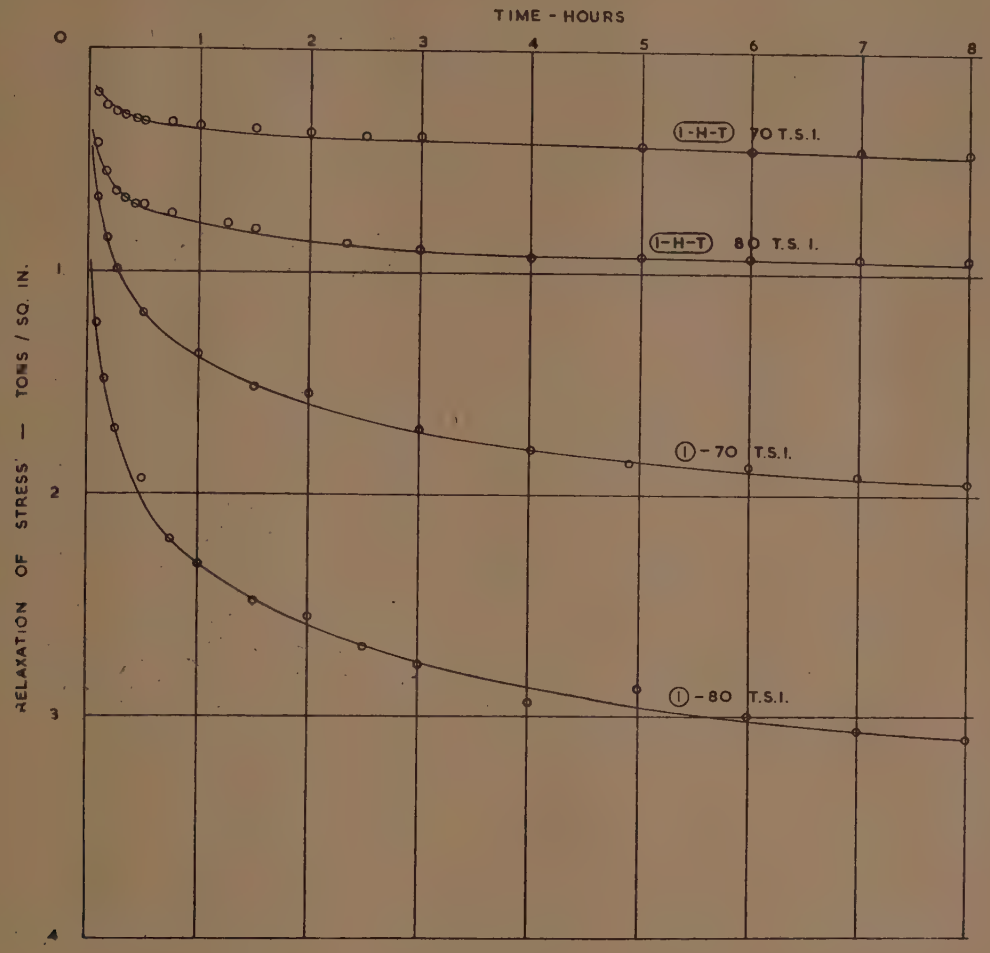


Fig. 12

alf their fire resistance, and likely to retain a high ercentage of their original strength on cooling but with marked residual deflection and loss of prestress." he reason for the retention of some of the original rength on cooling can be found in Fig. 3. An increase the temperature from about 240°C. to about 300°C. esults in an appreciable percentage increase in the rain in the heated wire, and in the residual strain after ooling.

Post-tensioned Wire

For a parabolic cable of span l and centre sag e, the ngth of the cable is

$$s = l + \frac{8e^2}{3l} \quad \ldots\ldots\ldots (7)$$

nd the horizontal force at each end of the cable is

$$H = \frac{wl^2}{8e} \quad \ldots\ldots\ldots (8)$$

Now $$\delta e = \frac{df_1}{ds} \cdot \delta s + \frac{df_1}{dl} \cdot \delta l \quad \ldots\ldots (11)$$

and $$\delta H = \frac{df_2}{ds} \cdot \delta s + \frac{df_2}{dl} \cdot \delta l \quad \ldots\ldots (12)$$

Substituting in equations (11) and (12)

$$\frac{\delta e}{e} = \frac{8e^2+3l^2}{16e^2} \cdot \frac{\delta s}{s} + \frac{8e^2-3l^2}{16e^2} \cdot \frac{\delta l}{l} \quad \ldots (13)$$

and $$\frac{\delta H}{H} = \frac{-(8e^2+3l^2)}{16e^2} \cdot \frac{\delta s}{s} + \left[2 - \frac{(8e^2-3l^2)}{16e^2}\right] \frac{\delta l}{l} \quad \ldots (14)$$

Consider the cable in a post-tensioned beam which has a span of 30 ft. and a centre sag of 3 in., then equation (13) becomes

$$\frac{\delta e}{e} = 2700.5\,\frac{\delta s}{s} - 2699.5\,\frac{\delta l}{l} \quad \ldots\ldots (15)$$

and equation (14) becomes

same. Hence from equation (15) and (16) the percenta increase in the sag and the percentage decrease in th

horizontal force would both be equal to $\frac{\delta s}{s}$.

Since the extension of the length of the wire whe heated is considerably increased with tensile stress, it

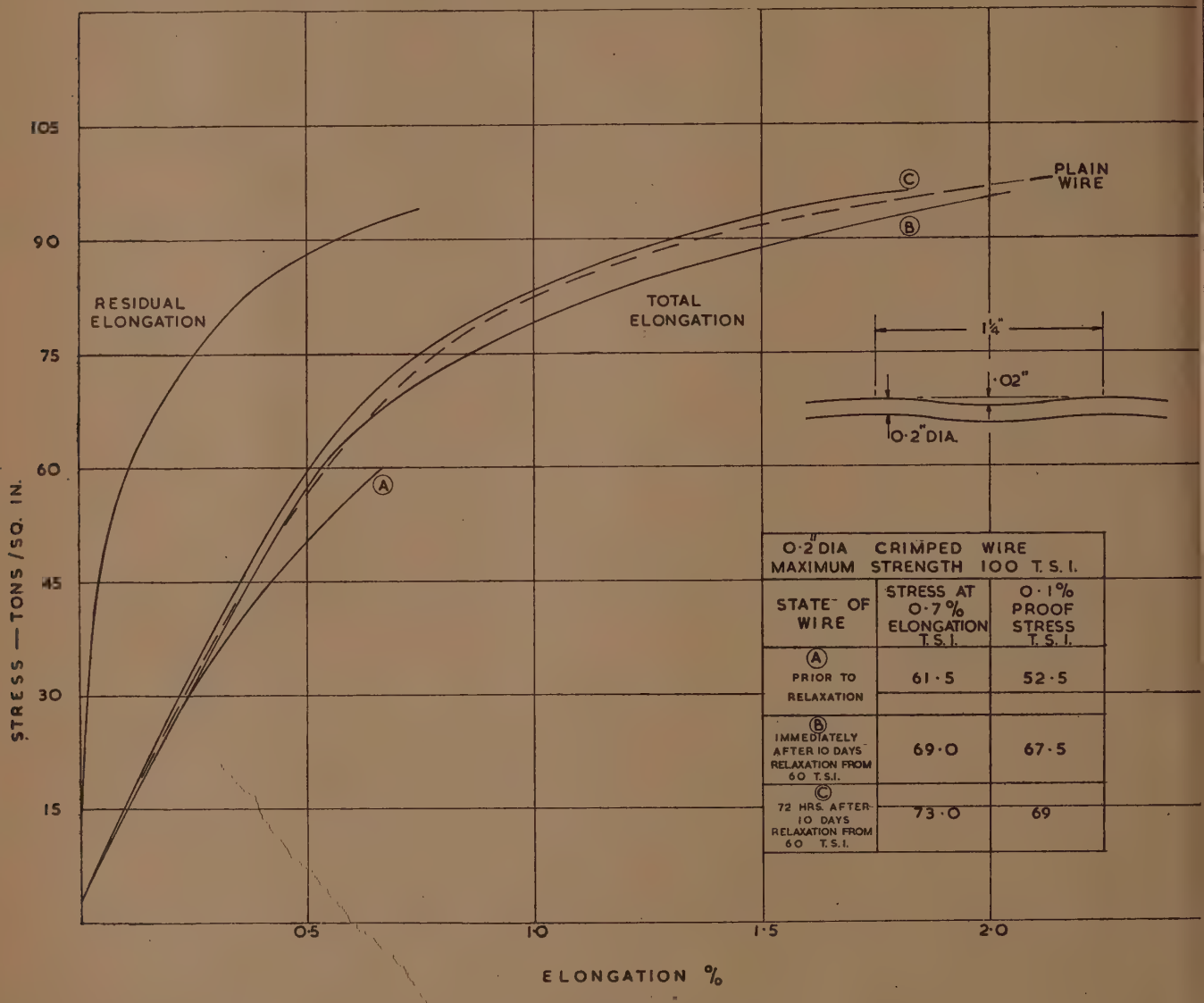


Fig. 13

$$\frac{\delta H}{H} = -2700.5\,\frac{\delta s}{s} + 2701.5\,\frac{\delta l}{l} \quad \ldots\ldots (16)$$

It is often supposed that the linear coefficient of expansion for concrete and steel are the same, when the

percentage increase in the length of wire $\left(\frac{\delta s}{s}\right)$

and in the span $\left(\frac{\delta s}{l}\right)$ would be approximately the

reasonable to suppose that the extension of the concret will be decreased with compressive stress. Suppose tha

$\frac{\delta s}{s}$ is 2 per cent. greater than $\frac{\delta l}{l}$ and that $\frac{\delta s}{s}$ is 1 per cen

Then from equation (15) $\frac{\delta e}{e} = 54\,\frac{\delta s}{s}$ and the sag in

creases to 4.62 in., and from equation (16) the horizonta force is reduced 52 per cent.

Hence if the longitudinal expansion of the beam is prevented, then $\frac{\delta s}{s}$ may be much greater than $\frac{\delta l}{l}$ and as the Fire Research Station found the beam may fail earlier than if it is free to expand.

The difference between $\frac{\delta s}{s}$ and $\frac{\delta l}{l}$ is likely to be increased on cooling because the compression in the concrete will tend to decrease the longitudinal length of the beam, while as shown in Table 5 there will be a residual strain in the length of the wire, and the beam will take on a permanent set.

(b) The relaxation of stress at 1 day, 1 week and if possible, 1 month and 1 year.

(c) The speed of relaxation-time curve for the first 24 hours.

(i) (a) Discloses any plastic behaviour, and records the change in slope above 60 per cent. maximum stress from which may be judged the ductility and the speed of elongation as the stress approaches the maximum.

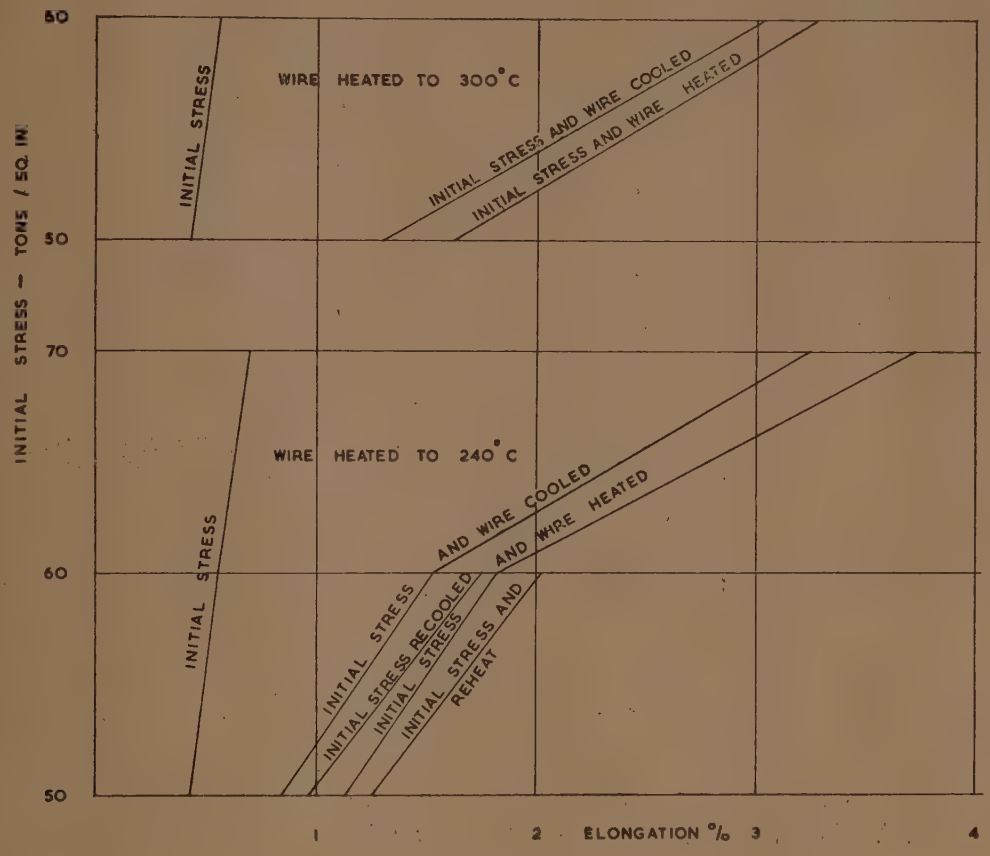


Fig. 14

Efficiency of Wire as a Prestressing Tendon

To assess the efficiency of a wire as a prestressing tendon it is necessary to determine the following facts from :

(i) Tension Test.
- (a) The nominal stress-total elongation curve.
- (b) The nominal stress-residual elongation curve.
- (c) The unloading stress-total elongation curve. from about 70 per cent. maximum stress.
- (d) The stress at 0.7 per cent. total elongation.
- (e) The total elongation at 70 per cent. maximum stress.
- (f) The total elongation at maximum stress.
- (g) The elongation after fracture.

(ii) Relaxation Test (Initial Stress 70 per cent. maximum stress).
- (a) The relaxation of stress-time curve for the first eight hours.
- (b) Indicates up to what stress the wire behaves elastically.
- (c) The loss of steel stress due to concrete deformations must be measured on the unloading curve and not on the loading curve. These two curves are shown in Fig. 1 for Wire (1).
- (d) The higher the value of this stress the less plastic the behaviour of the wire, also the loss of stress due to concrete deformations is likely to be smaller.
- (e) This is a measure of the elongation at about the preliminary stress, the higher this value the easier it is to measure and to use as a check on the load in the wire during the prestressing operation.
- (f) The difference between this elongation and the previous one (e) is a measure of the deformation that the wire will stand if it becomes overloaded.

Desirable Characteristics of a Wire

The ideal wire would have elastic characteristics which are stable, so that the future behaviour of the prestressed concrete in which it is used can be predicted with some certainty. After stressing to a point below

the elastic limit, the stress-strain relationship should be reversible, and the residual elongation should be zero. The slope of the straight line part of the stress-strain diagram should be as small as possible up to a high proportion of the maximum stress. The small slope would result in large deformations at the preliminary stress which, as previously indicated will result in smaller stress losses due to concrete deformations. Since, if the material is elastic, the loading and unloading curves coincide :

The elongation at the maximum stress should be as large as possible so as to give a good warning of failure. If the wire is overloaded either during the prestressing operation or when it is acting as a prestressing tendon in a beam, it is desirable that the difference in the elongation at maximum stress and at a preliminary stress of 70 t.s.i. should be as high as possible. For Wire (1) the ratio of these elongations is about 4.7.

The slope of the curved part of the stress-strain diagram should not change abruptly, but should have a gradual continuous decrease from the end of the elastic line up to the point of maximum stress. If the change in slope is too abrupt, the stress in the wire may be close to the maximum while the elongation is still comparatively small, and although the elongation at the maximum stress may be high, in event of overloading the degree of warning of failure would be insufficient, and would result in abrupt failure ; whereas if the change in slope is more gradual, the approach to the maximum stress is disclosed by earlier and larger elongations at stresses well below the maximum.

Generally the aim should be towards larger diameters, with high maximum stress, and low relaxation of stress. The latter should be rapid in the first hour of stressing, and then rapidly become constant.

Conclusions

1. Wire in the as-drawn condition possesses non-elastic characteristics ; the deformations are not reversible and the loading and unloading curves form loops.

2. After being subjected to a number of repetitions of tensile load, the characteristics of the as-drawn wire tend to be more elastic ; but the non-elastic characteristics return after the wire has been subjected to some bending.

3. Due to the plastic character of the as-drawn wire, it can be subjected to severe deformations without seriously increasing the load.

4. The proof stress does not give any indication of the behaviour of wire as prestressing tendon, and it is suggested that the stress at say 0.7 per cent. elongation be substituted. The proof stress is dependent on the curvature of the stress-strain diagram which is subject to the rate of loading effects.

5. Ductility is defined as the uniform elongation which takes place up to the maximum load, i.e., the amount of deformation which can be applied in service before failure. This ductility can be measured in terms of the reduced diameter at a point 10 dia. away from the point of fracture.

6. The relaxation of stress varies even for wire in the as-drawn condition, and depends on the particular conditions which occurred both before, and during the cold drawing process. The relaxation is at first rapid ; and the relaxation from an initial stress of 70 t.s.i. after one hour is approximately 50 per cent. of the relaxation after 10 days.

7. Artificial strain-ageing of the as-drawn wire at an optimum temperature results in a wire with elastic characteristics, and the deformations are reversible from normal service stresses. The ductility is improved and there is no residual elongation from a stress below 75 t.s.i. Lubricant on the surface of the wire is also removed by heating.

8. The loss of steel stress due to concrete deformation from 70 t.s.i. is smaller for the strain-aged wire than for the wire in the as-drawn condition.

9. The effect of strain-ageing on the relaxation is indirect and the relaxation is smaller than for the as-drawn wire if the initial stress is not greater than 60 t.s.i.

10. A wire strain-aged under tension not only possesses elastic characteristics, but smaller relaxations of stress than do either as-drawn, or strain-aged wire, even from an initial stress of 80 t.s.i.

11. The characteristics of drawn wires are not a simple function of either diameter or maximum strength but are dependent on the basic material and its treatment, and the extent and manner of subsequent cold reduction and ageing. The relaxation of such wires is not related to elastic characteristics, or the maximum strength or elongation at this stress. It would be unwise to extrapolate in Figs. 8, 9 or 10 because at the end of long periods it is unknown whether the rate of relaxation will increase, or whether the wire will fracture.

12. The strain in a stressed wire is considerably increased by the application of temperatures such as would occur in a fire.

Acknowledgements

The work was carried out by the author in the Engineering Laboratories of the University College, Cardiff where he will carry out further investigations into the characteristics of wire manufactured by cold drawing and other methods. He acknowledges with gratefu thanks the valuable assistance of the technical staff, and in particular that given by Messrs. A. H. Randle and C. A. Hopkins.

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Discussion

The Literature Committee would be glad to consider the publication of correspondence in connection with the above paper. Communications on this subject intended for publication should be received by October 1st, 1953.

Concrete Beams with Compression Steel

A QUICK METHOD OF DESIGN

By F. H. Blake, M.I.C.E., M.I.Struct.E.

The presence of the slab on the compression side of a -beam permits the use of a large amount of tensile eel in the rib. In continuous spans, however, the verse moment at the support is usually comparable in agnitude with the span moment, and so also calls for a nsiderable amount of tensile steel ; but as the slab now not on the compression side the rib concrete ten requires the assistance of compression reinforce- ent, Fig. (i).

The rib size of a continuous beam is thus frequently ntrolled by the compression requirements at the pports, and the design problem is usually to find the mpression steel required, with b and d provisionally nown. In rectangular beams such as crane and gantry rders, and precast beams where it is important to eep the size down, the same problem may occur at mid- pan Fig. (ii). Compression steel is of course required

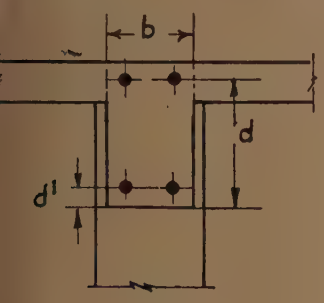


Fig. (i)

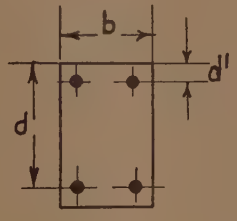


Fig. (ii)

n any rectangular beam when the tensile steel exceeds he "economic" percentage.

A method often used by designers involves finding the esistance moment of the beam without compression teel, using the appropriate value of $Q = M/bd^2$ for the ermitted stresses and then calculating the compression teel required to make up the balance of resistance. For this purpose an approximate lever arm is estimated, nd the stress in the compression steel is either judged r computed from its distance from the neutral axis. Finally, the tensile steel is calculated using its appro- riate lever arm. Although the result is sufficiently ccurate for practical purposes, the operations take everal minutes to perform in spite of the approxima- ions made.

Another method, possessing the merit of simplicity, s the Steel Beam Theory. In this, equal top and ottom steel is provided and assumed to be stressed to he maximum permissible value in tension, e.g., 18,000 si., the lever arm being taken as the distance between he reinforcements or about $0.85d$. This method used o be popular in competitive design because it always ave a smaller beam, but owing to the higher concrete tresses now permitted it no longer enjoys that advan- tage in many cases, as may be seen from the Table below.

The correct expression for resistance moment of a doubly-reinforced rectangular section contains several variables and is too complex for practical design. Graphs of the neutral axis ratio n_1, and lever arm ratio a_1, in terms of the steel ratios $p = \frac{At}{bd}$ and $p' = \frac{Ac}{bd}$ are given in certain textbooks for particular values of d'/d, and a nomogram recently published* enables n_1, to be read off for any value of d'/d ; but the equations for resistance moment, or for the multiplier $Q = M/bd^2$, viz.,

$$Q_s = tpa_1$$

$$Q_c = c\left[n_1 \left(\frac{1}{2} - \frac{n_1}{6}\right) - \frac{p'}{n_1}(m-1) \left(n_1 - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]$$

are still much too cumbersome for practical use.

It is possible, however, for given values of c and d'/d, to plot Q as a simple chart for all combinations of p and p'. The modular ratio m is now stabilised at 15 (except for water-retaining structures). Moreover, Q does not vary rapidly with changes of d'/d.

The four accompanying charts are based on $m = 15$ and $t = 18000$ psi. They cover two values of d'/d,

Example : 18" by 9" Beam ; assumed $d = 15\frac{1}{2}$", $d^1 = 2\frac{1}{4}$".

Resistance Moment in/lb.	c lb./in)		Reinforcement % by Chart or calculation	Reinforcement % by Steel Beam Theory
700,000	1000	p	2.14	2.11
		p^1	1.73	2.11
,,	1250	p	2.16	2.11
		p^1	0.61	2.11
900,000	1000	p	2.75	2.71
		p^1	2.89	2.71
,,	1250	p	2.75	2.71
		p^1	1.50	2.71

namely 0.15 and 0.10, and two values of c, namely 1000 psi. and 1250 psi., which values under B.S.C.P. 114 correspond to the two mixes most commonly used in practice, namely, 1 : 2 : 4 and 1 : $1\frac{1}{2}$: 3. As explained later, these charts can in fact be utilised for the whole range of normal design requirements.

The value of Q is on the safe side so long as the actual depth of the compression steel is less than the ratio shown, and the error is small. The ratio 0.10 applies only to beams over about 24 in. deep, but the ratio 0.15 is very close for the majority of other cases. The ratio naturally depends on the bar diameter and number of rows, and in a beam heavily reinforced in relation to its depth d'/d may exceed 0.15. For example, in a section $13\frac{1}{2}$ in. deep overall carrying two rows of 1 in. bars top and bottom it would reach 0.20 ; and for that ratio the 0.15 charts could be used with the Q values reduced by 2 per cent. at the bottom ranging to 8 per cent. at the upper end of the chart. Even in this rather

*A. Feldman, CONC. AND CONSTRUCTIONAL ENG., March, 1949.

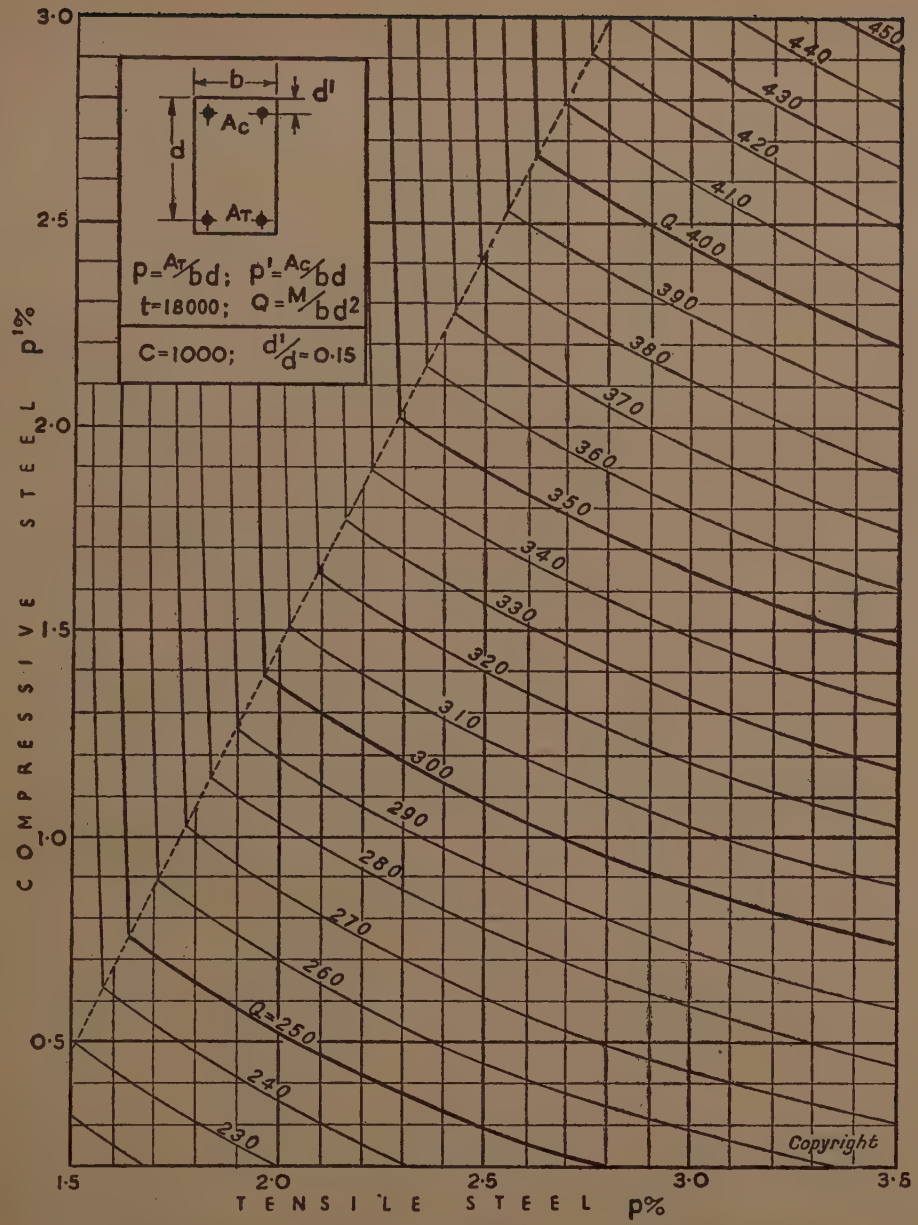


Chart A

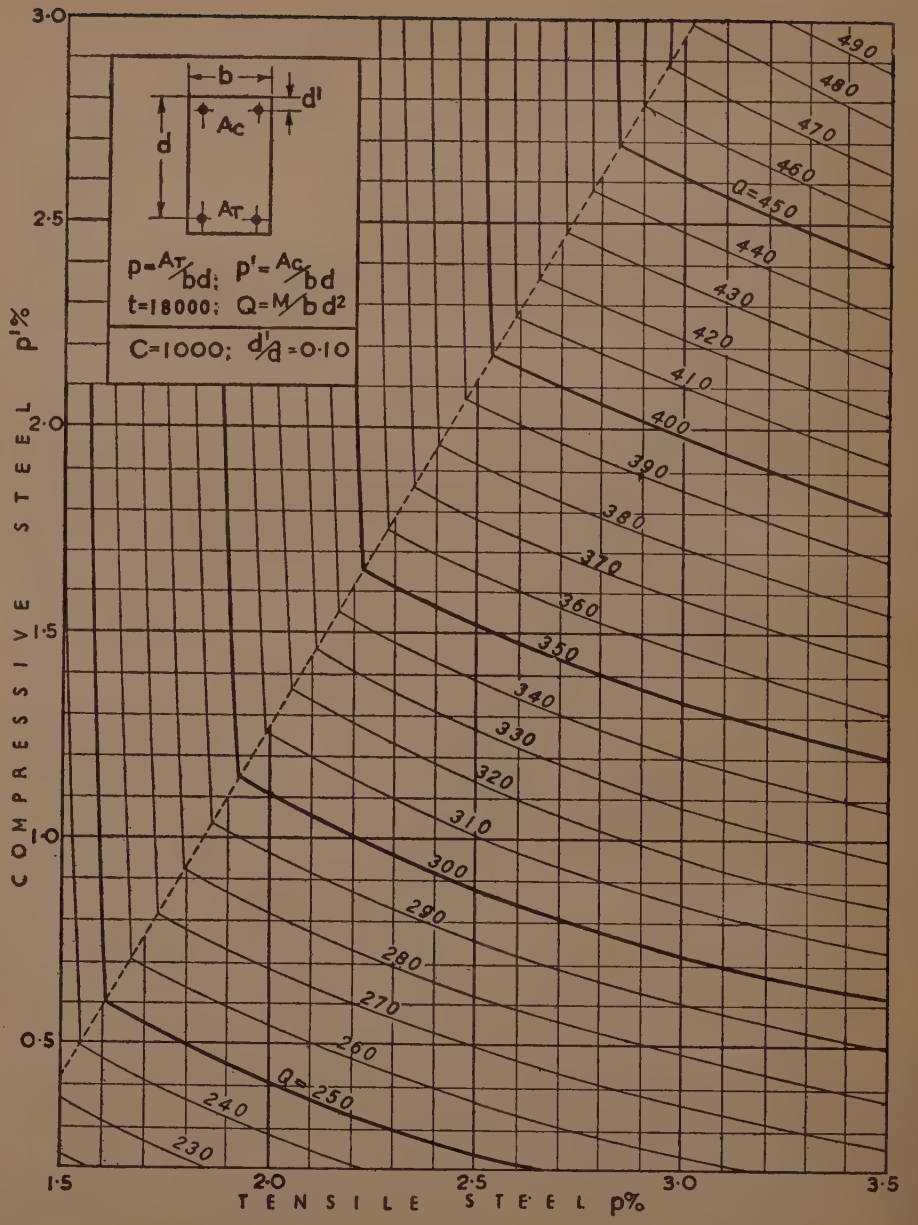


Chart B

DOUBLY REINFORCED BEAMS—Charts of Q for c = 1000

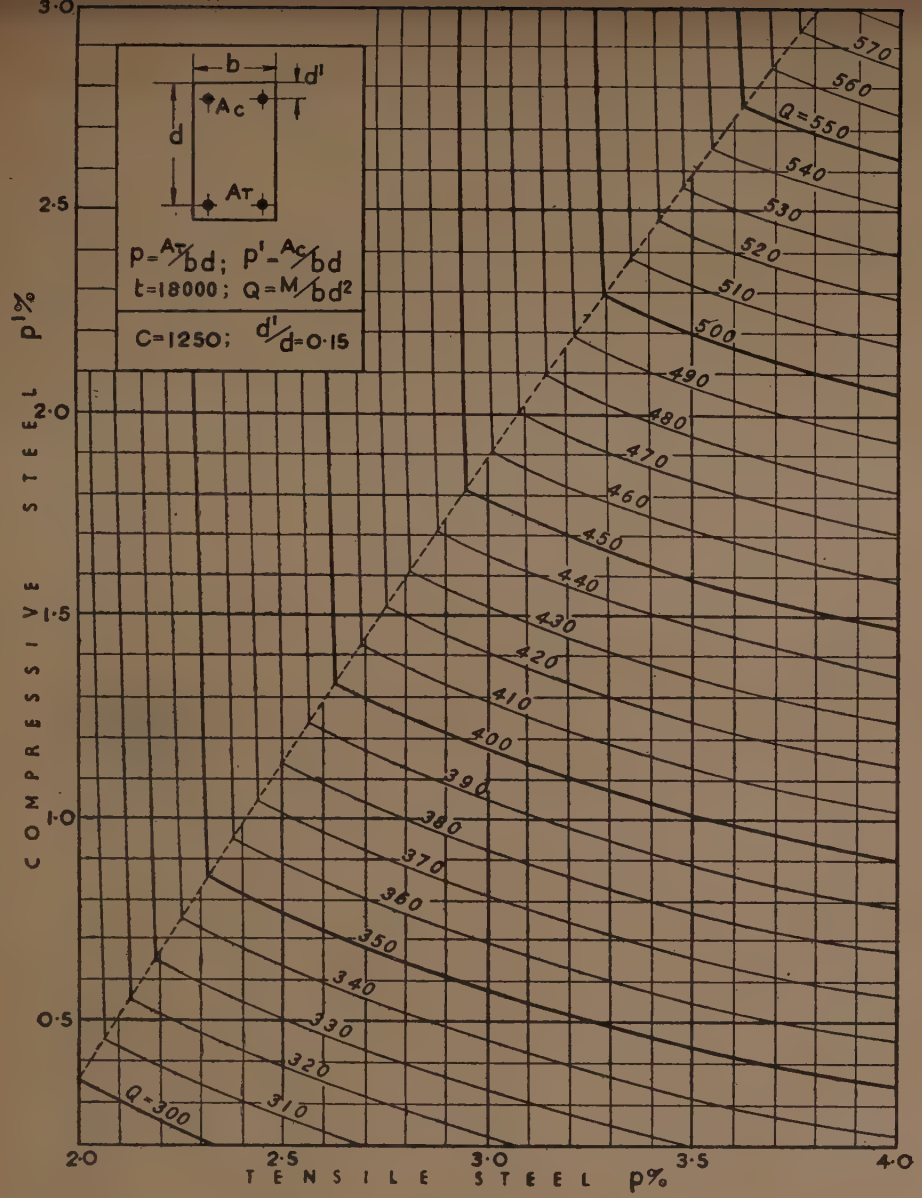


Chart C

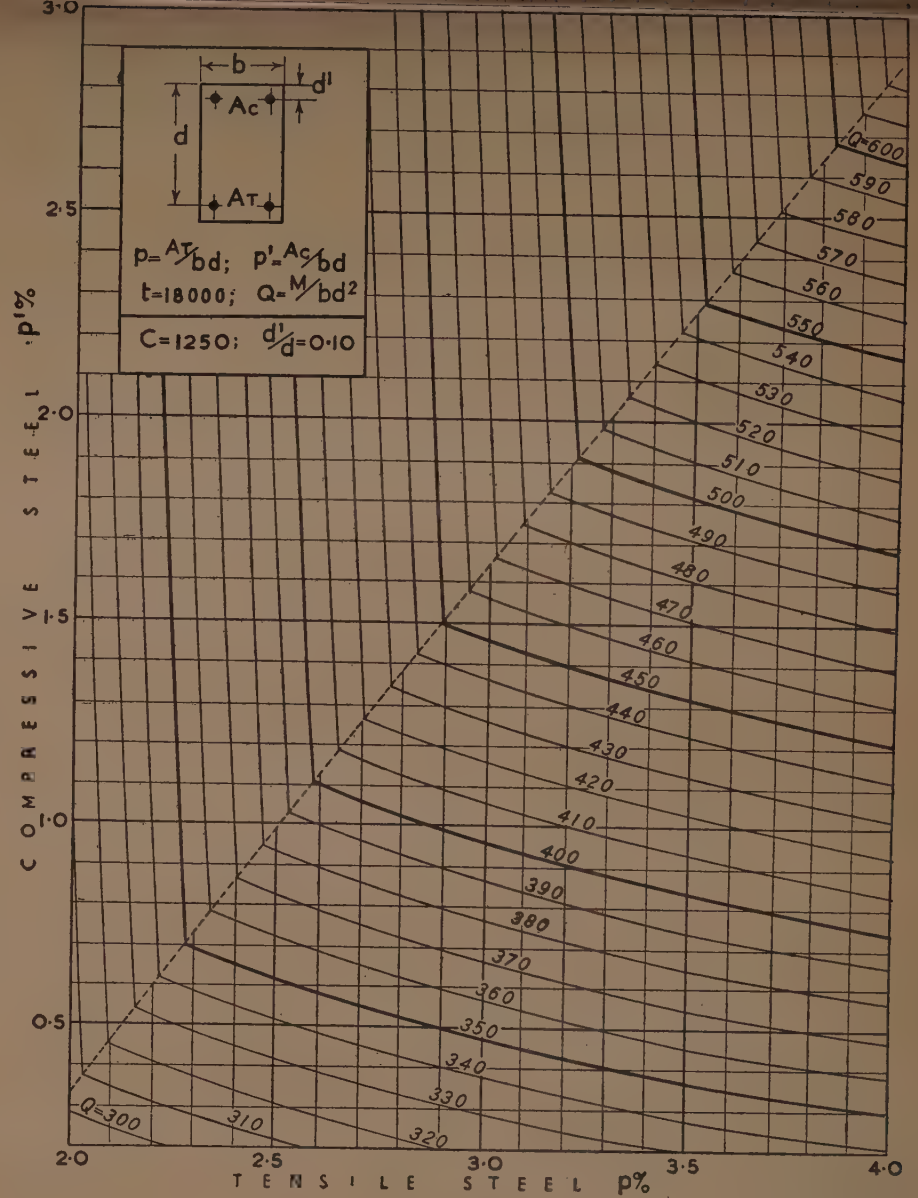


Chart D

DOUBLY REINFORCED BEAMS—Charts of Q for $c = 1250$

extreme case the adjustment, made by eye, will still give a result nearer the truth than the orthodox rough calculation.

In these charts the flat curves are plotted from the equation for Q_c and the steep curves from that for Q_s ; they therefore involve no approximations as to lever arm or compressive steel stress. At all points along the dotted line both materials will be stressed to their permitted values. To the right of the dotted line the concrete is working at its maximum permitted stress, while to the left the tensile steel is the limiting factor, being stressed to 18,000 psi. It will be seen that in this zone large increases in compressive steel make little difference to the strength.

The method of use is to compute the required value of Q from M/bd^2 and read off p and p' for that value of Q at the dotted line. The chart thus gives both tensile and compressive steel requirements at once. Any pair of values of p and p' giving the required Q would be satisfactory, but greatest economy is to be expected when $A_T + A_C$ (or $p + p'$) is a minimum, and this occurs at the dotted line. The approximate design method previously mentioned gives results in the vicinity of the dotted line ; designing by the chart however is much quicker and less liable to error, giving the answer as rapidly as the Steel Beam method, while obviating the dubious assumptions of that method and generally gaining something in economy.

Working stresses differing from 1000 or 1250 psi. are sometimes required, e.g., when utilising the 10 per cent. increase allowed for vibrated concrete or when a reduction for slenderness is imposed. The charts may readily be used for such modified stresses because the values of Q in the flat curve zone (curves of Q_c) are directly proportional to c, while the Q_s curves (above the dot line) are practically independent of c. For example, use the chart of $c = 1000$ when the permitted c 1100 :—Compute Q as before, and suppose it is 3 Locate the $Q = 300$ curve at the dotted line. T steep part of the curve is that of Q_s, which is unaffec by the change in concrete stress ; produce it downwa

$$\text{to meet a } Q_c \text{ curve equal to } \frac{10}{11} \times 300 = 273. \quad (1$$

latter as shown on the chart is for $c = 1000$, and $c = 1100$ its value would be 300.) This point is the required and gives the requisite p and p' with qu sufficient accuracy for normal design purposes.

For a concrete stress less than that of the chart, as example 900 psi, a similar procedure is followed, but that case if the required Q is again 300, the Q_c cu

$$\text{corresponding to } \frac{10}{9} \times 300 = 333 \text{ is taken and extend}$$

beyond the dotted line into the steel zone to meet line of $Q_s = 300$. As before, any pair of values o and p' corresponding to the extended curve $Q_c =$ would also be suitable.

An advantage of the charts is that the desirability varying either A_T or A_C may be seen at a glance. I example, A_T is usually provided, in part at least, span bars bent up over the support and the near suitable bar arrangement often provides an area steel substantially larger than caculation requir The curves show that in such a case the compress steel could be reduced, and the corrected amount n be read off direct.

The Use of Deformed Steel Bars in Reinforced Concret

The results of an investigation by an ad hoc *committee of the Science and Research Committee are published for the information of members.*

*The Council is confident that the investigation will be of considerable value to the Committee who are in process of revising Code of Practice C.P.*114, *" The structural use of normal reinforced concrete in buildings."*

Introduction

In reinforced concrete design it is generally assumed that the strain in the steel reinforcement is equal to the strain in the concrete immediately surrounding it. This is substantially true in zones of compression but, in the parts subject to tension, the working stress in the steel is usually high enough for the strain to be more than the surrounding concrete can withstand without cracking. The size of the cracks formed in this way and their spacing along the reinforcement depends primarily on the bond but is also affected by the strain in the steel, the tensile strength of the concrete and its modulus of elasticity.

The stresses adopted in designing reinforced concrete structures using plain bars are limited not only by the strength of the steel but by the need to keep the width of the cracks in the tension zone small enough to avoid corrosion of the steel.

Insufficient information is available to allow a correlation between crack widths, bond strength and steel stress ; but it is thought that any substantial increase in steel stresses would not be possible in exposed str tures without dangerously increasing the width of cracks unless a greater degree of bonding can be obtain

The use of deformed bars in place of plain bars usua permits higher steel stresses to be employed, since bond characteristics tend to lead to the formation more numerous but finer cracks. These higher stres can often be achieved by merely substituting for pl bars an equivalent cross-sectional area of deform bars. Other factors have to be considered, howev including the strength and elastic properties of the st and the possibilities of increased deflection in cert members. For example, the use of higher steel stres may lead to excessive deflection unless the effect depth of members is increased.

Where it is proposed to use deformed bars in place plain bars all these factors require consideration.

It should be emphasised that there has been lit experience of the use of deformed bars, other than c twisted bars to B.S.1144, in this country, and that has been necessary to rely on knowledge gained in ot countries.

Deformed Bars

(1) Definition of Deformed Bars

A deformed bar is any steel bar of material conform to clauses (3) and (4) and having surface deformati or being so formed as to develop a greater bond streng when embedded in concrete, than that which would developed by an equivalent plain round steel bar.

Types of Deformed Bars

wo types of deformed bars are considered ;—

ype (A) Bars having deformations conforming with requirements of the American Society for Testing erials Standard A305-50 T, as given in Appendix I,

or

ars which can be shown by test (see Appendix III) possess bond characteristics not inferior to those of s complying with the requirements of Appendix I.

ype (B) Cold twisted steel bars complying with the uirements of B.S. 1144, " Cold twisted steel bars."

Steel for Deformed Bars

Steel for Bars of Type (A)

General

Deformed bars of Type (A) may be either hot rolled s having a definite yield stress or cold worked bars which a proof stress has to be defined.

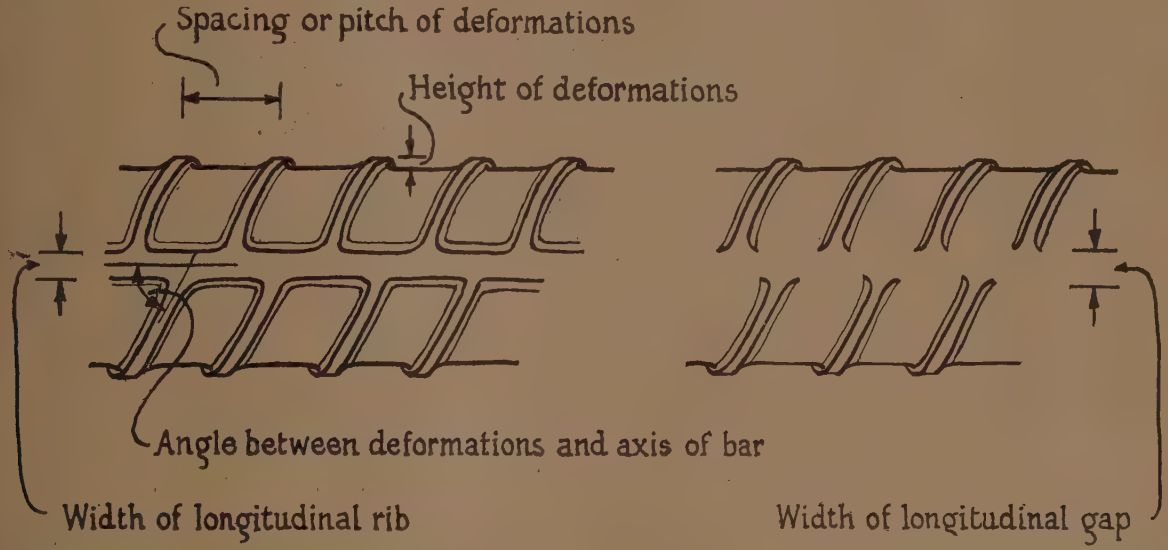

Spacing or pitch of deformations
Height of deformations
Angle between deformations and axis of bar
Width of longitudinal rib
Width of longitudinal gap

Fig. 1.—Diagrammatic sketch of a deformed bar showing some of the principal dimensions mentioned in Appendix 1

Note.—The bar may or may not have a longitudinal rib and the deformations may or may not be staggered on each side of the rib. In addition, the direction of the deformations may be alternately reversed or they may be reversed on opposite sides of the longitudinal rib

Since at a given steel stress the width of cracks tends be smaller with deformed bars than with plain bars, gher working stresses are permissible in deformed bars ovided their strength properties are adequate.

) Hot rolled deformed bars

High tensile steel complying with B.S. 785, " Rolled eel bars for concrete reinforcement," with a yield stress approximately 50,000 lb./sq. in., is adequate for plain rs where working stresses have to be limited to avoid e danger of cracking.

For deformed bars, however, a higher yield stress of t less than 60,000 lb./sq. in. for bars over 1 in. in ameter or 70,000 lb./sq. in. for bars up to 1 in. in ameter is considered advisable.

Cold Worked deformed bars

For cold worked bars the stress corresponding to an ngation of 0.5 per cent. of the gauge length may be cepted as " proof stress "* in accordance with B.S. 44. This is a safer limit than the yield stress of hot lled bars, since a structure in which the stress has ached this limit is still capable of carrying further load.

*In the current (1943) edition of B.S. 1144, cl. 9, this stress is ferred to as " yield stress."

The minimum values specified in B.S. 1144 for the proof stress* of single twisted bars are as follows :—

For bars under $\frac{3}{8}$ in. diameter or thickness ... ... 70,000 lb./sq. in.

For bars $\frac{3}{8}$ in. and over in diameter or thickness ... ... 60,000 lb./sq. in.

It is suggested that for cold worked deformed bars these limits should be changed as follows :—

For bars up to and including $\frac{1}{2}$ in. diameter ... ... 75,000 lb./sq. in.

For bars exceeding $\frac{1}{2}$ in. diameter... 66,000 lb./sq. in.

It is an essential condition for this steel that brittleness be avoided. For this reason it is considered that cold-worked bars should comply with the following requirements which are not at present specified in any British Standard :—

(i) There should be an adequate margin between proof stress and ultimate stress. Further experience in the use of cold worked bars is necessary before the minimum margin between proof stress and ultimate stress can be closely specified, but it is considered that in no case should the proof stress exceed $\frac{7}{8}$ of the ultimate strength.

(ii) The " uniform elongation," i.e., the permanent elongation which occurs in a part of the specimen not affected by " necking " should not be less than 4 per cent.

(iii) The bar should satisfy the bend test given in Appendix II.

It should be noted that, with some steels, brittleness varies considerably with temperature. Such steels, even though satisfactory when tested at normal temperatures, may prove difficult to bend at very low temperatures.

(4) Steel for Bars of Type (B)

Reference should be made to B.S. 1144, " Cold twisted steel bars."

Design

(5) General

It is considered that, provided the concrete used satisfies the requirements of Code of Practice C.P. 114

(1948), Tables 1 and 2, the basis of design given in that Code is applicable to the deformed bars covered by this investigation, with the following modifications :—

(*a*) *Design of Sections*

The substitution of deformed bars for mild steel can be made without modification of the concrete section, provided the deflection does not become excessive.

Where the section is designed on the basis of mild steel reinforcement, the substitution of deformed bars may lead theoretically to a compressive concrete stress higher than that recommended in C.P. 114 (1948). Research has shown however that within the range of the yield or proof stress envisaged in clauses (3) and (4) the load factor against ultimate failure is not reduced by such substitution.

(*b*) *Permissible tensile stresses*

(i) The maximum tensile stress in deformed bars of Type (A) should not exceed one-half the yield or proof-stress, or 33,000 lb./sq. in., whichever is the less. In slabs, however, the limit of 33,000 lb./sq. in. may be increased to 37,000 lb./sq. in.

(ii) For bars of Type (B) the maximum tensile stress should not exceed 27,000 lb./sq. in. or, in slabs, 30,000 lb./sq. in.

Where the increased stresses in slabs given in (i) and (ii) above are used, it is considered advisable that the span/depth ratio should not exceed 80 per cent. of the value given in Table 10 of Code of Practice C.P 114.100 (1950), " Suspended concrete floors and roofs."

(*c*) *Permissible bond stresses*

The permissible bond stresses in deformed bars may exceed those allowed in C.P. 114 (1948) for plain round bars by the following percentages :—

Type (A) deformed bars ... 50 per cent.
" (B) " " ... 10 per cent.

The Committee is satisfied that the above bond stresses can be used without increasing the minimum cover recommended in C.P. 114 (1948).

(*d*) *Tolerances in bar diameters*

Where bars are supplied with a tolerance both above and below the nominal diameter then the stresses should be estimated on the basis of the minimum diameter permitted and not the nominal diameter.

(*e*) *Anchorage*

With deformed bars it may be unnecessary and ma even be undesirable to use hooks. For bars of Type (A in tension the minimum value of " n " given in C.P. 11 (1948) cl. 307, can be reduced from 12 to 8, and for Typ (B) bars from 12 to 10.

(*f*) *Bends in bars*

The radius of curvature of bends in bars should no be less than eight times the diameter of the bar.

APPENDIX I

Deformed bars Type (A)—Specified deformations extracted from A.S.T.M.S. A305-50 T (See als American Society for Testing Materials Standard Fig. 1)

(a) Deformations shall be spaced along the bar a substantially uniform distances. The deformations o opposite sides of the bar shall be similar in size and shap

(b) The deformations shall be placed with respect t the axis of the bar so that the included angle is no less than 45 degrees. Where the line of deformation forms an included angle with the axis at the bar of fro 45 degrees up to and including 70 degrees, the deforma tions shall alternately reverse in direction on each side or those on one side shall be reversed in direction fro those on the opposite side. Where the line of deforma tions is over 70 degrees, a reversal in direction is no required.

(c) The average spacing or distance between deforma tions on each side of the bar shall not exceed seven tenths of the nominal diameter of the bar.

(d) The overall length of deformations shall be suc that the gap between the extreme ends of the deforma tions on opposite sides of the bar shall not exceed 12½ per cent. of the nominal perimeter of the bar Where the extreme ends terminate in a longitudinal rib the width of the longitudinal rib shall be considered a the gap. Where more than two longitudinal ribs ar involved, the total width of all longitudinal ribs sha not exceed 25 per cent. of the nominal perimeter of th bar ; furthermore, the summation of gaps shall no exceed 25 per cent. of the nominal perimeter of the bar The nominal perimeter of the bar shall be 3.14 times th nominal diameter.

TABLE 1.—Dimensional Requirements for Deformed Steel Bars for Concrete Reinforcement

Deformed bar designation number (a)	Unit weight lb. per ft.	Nominal dimensions, round sections			Deformation requirements		
		Diameter in.	Cross-sectional area sq. in.	Perimeter in.	Maximum average spacing in.	Minimum height in.	Maximum gap (chor of 12½ per cent. nominal perimeter in.
*2½	0.261	0.31	0.08	0.982	0.219	0.013	0.120
3	0.376	0.375	0.11	1.178	0.262	0.015	0.143
*3½	0.511	0.44	0.15	1.375	0.306	0.017	0.167
4	0.668	0.500	0.20	1.571	0.350	0.020	0.191
*4½	0.850	0.56	0.25	1.767	0.395	0.023	0.216
5	1.043	0.625	0.31	1.963	0.437	0.028	0.239
6	1.502	0.750	0.44	2.356	0.525	0.038	0.286
7	2.044	0.875	0.60	2.749	0.612	0.044	0.334
8	2.670	1.000	0.79	3.142	0.700	0.050	0.383
9	3.400	1.125	1.00	3.544	0.790	0.056	0.431

(a) Bar numbers are based on the number of eighths of an inch included in the nominal diameter of the bars. The nominal diameter of a deformed bar is equivalent to the diameter of a plain bar having the same weight per foot as the deformed bar.

(e) The average height of deformations shall be not s than the following percentages of the nominal ameter of the bar :—

eformed bar designation number	Minimum height of deformations : percentage of nominal diameter of bar
*2½	3½
3	4
*3½	4
4	4
*4½	4½
5	4½
6 and over	5

(f) The spacing, height and gap of deformations shall nform to the requirements prescribed in Table 1.

*These bar sizes have been interpolated by the Committee to rrespond with bar sizes used in this country.

APPENDIX II

Bend Test for Cold-worked Deformed Bars

Test. The specimen shall withstand without fracture, ing bent by pressure to a U-shape, round a pin the ameter of which is six times the diameter of the bar. he test shall be made at a bar temperature of between °F. and 70°F.

Test Pieces. The specimen shall be of sufficient length ensure free bending.

Apparatus. The apparatus used for carrying out the st shall be such as to provide for the following :—

1. Continuous and uniform application of force roughout the duration of the bending operation.
2. Unrestricted movement of the specimen at points contact with the apparatus.
3. Close wrapping of the specimen around the pin or andrel during the bending operation.

APPENDIX III

Measurement of Bond

) General

There is an essential difference in the mechanism of ond failure between the normal smooth round bar and e deformed bar. With the smooth bar the bond ress at a point on the bar builds up to a maximum lue and then decreases ; this action progresses along e bar and final failure takes place by crushing of the ncrete in contact with the bar and its consequent ithdrawal from the concrete. With deformed bars, owever, the contact surface is such that mechanical edging takes place between the concrete and the eel ; the bond stress rises to high values and final ilure is generally by bursting, in tension, of the concrete ver on the steel. The deformed surface specified in ppendix I is of the type that consistently produces the echanical wedging necessary for high values of bond ress.

) Comparative Tests of Bond Characteristics

The specification for surface deformations given in ppendix I is the result of an extensive investigation rried out in the U.S.A. Deformed bars not complying ith this specification should be subjected to a series of sts to demonstrate that their bond properties are not inferior to those of a bar complying with the requirements of Appendix I.

The following tests may be used for the purpose of the comparison and should be made on at least three specimens of the proposed new bar and three control specimens complying with the requirements of Appendix I.

The technique of bond testing is not yet sufficiently well established to enable standard details for these tests to be laid down. It is, however, essential that conditions of testing should be identical throughout each series of comparative tests. The concrete strength at the time of testing should be between 3,000 and 4,000 lb./sq. in. The concrete quality should be carefully controlled throughout the series of tests and the mix and cube strengths of the concrete recorded.

(*a*) *Pull-out test*

The bar should be embedded vertically in a block of concrete providing a cover of about three bar diameters to the steel. The length of embedment should be twelve diameters. The concrete block should be adequately reinforced against bursting by either independent steel stirrups or helical binding of sufficient diameter and pitch, placed approximately midway between the axially embedded bar and the outer surface of the concrete.

The test specimen should be mounted in a suitable testing machine in such a manner that the bar is pulled axially from the block. The end of the bar at which the pull is applied must be that projecting from the top face of the block as cast. A packer of rubber or similar material with a thickness of approximately ¼ in. should be placed between the concrete surface of the block and the surface of the test rig. The rate of loading should be the same for each test.

A suitable dial gauge should be attached to the unloaded end of the steel bar with the spindle in contact with the concrete in such manner that the gauge records the relative slip. The load at which a relative slip of one-twentieth of an inch takes place should be recorded for the purpose of comparison between the types of bar under test.

(*b*) *Beam tests*

Tests of this type depend upon the available facilities but will provide valuable information concerning crack formation and spacing, etc. The size of beam will be controlled by the size and capacity of the test frame although tests may be devised in which pairs of beams are coupled together at the ends and the load then applied by jacks placed between the beams. "Four-point" loading has most frequently been used for tests of this type.

The section of the beam should be designed to suit the purpose of the test and ensure that the beam does not fail by compression of the concrete or diagonal tension before the comparative bond characteristics of the bars are made evident by the width and spacing of the cracks.

(*c*) *Embedded rod test*

In this test a steel bar embedded axially in a concrete prism is subjected to a tensile pull applied at each of the projecting ends. Provided the test can be carried out on a sufficient length of bar, comparative information may be obtained on the bond characteristics of different kinds of surfaces. It may be stated that the results of tests of this type are not generally conclusive but the test results are often worth consideration in conjunction with tests (*a*) and (*b*).

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed

To the Editor of THE STRUCTURAL ENGINEER.

Prestressed Concrete Bridges and other Structures

By DONOVAN H. LEE, December, 1952

Sir—The writer found this paper of the greatest value and would be glad if the author could amplify some points.

A reference to an improved type of pile head was made on page 306, and details of this would be useful ; it had occurred to the writer that the disintegration often seen was due to the impact not being applied simultaneously to the concrete and the bars, and that the cost of an improved head would be offset by the great increase in rate of driving possible with heavier blows, and found with the Franki and other types of mandrel pile.

The writer could not agree with the statement on page 306 that distributing the bars around the periphery gave any improvement in moment resistance ; the same statement was also seen repeated elsewhere in regard to beams, but the reverse appeared to be correct to the writer. If the distributed bars were to resist the same ultimate moment as the same bars clustered near their centroid, they would all have to reach their ultimate stress simultaneously, but this could not be the case, as those in the compression zone would actually decrease in stress : the clustered bars were therefore more efficient, and this applied at working stress as well as at ultimate load, as the working stress was usually higher than the proportional limit of the steel.

The design shown in Fig. 21 appeared to apply load at the extreme edge of the girder flanges, and to apply torsion to the girders, resisted by struts : it seemed to the writer that the struts would add further load to the edge of the flanges, and the arrangement appeared dangerous. The writer had been impressed by the extreme weakness of thin sections in torsion (unless they were box-shaped). The weakness was not due to overstress, but to the small rigidity of such members in torsion, and the struts would be called on for all such stress resistance.

The variation in stress at the level of the cables was stated on page 310 to be only 500 lb./sq. in., but this could only apply to special cases such as composite beams. In normal prestressed beams the initial stress much exceeded 2000 lb./sq. in., and the stress under full load was tensile, and the variation at the level of the cables could easily be 2000 lb./sq. in. In the secondary beams at London Airport this variation was 1900 lb./sq. in., with 15 in. " cover " to the bars, and the variation would be larger with small cover.

The variation in steel stress would be at least four times this, and probably more, and with bars stressed to 80,000 lb./sq. in. the variation in stress due to live load would be over 10 per cent., but with wires stressed to 120,000 lb./sq. in. it would be reduced, but still could exceed 7 per cent.

In composite beams a large part of the superimposed load was due to the cast *in situ* deck, as on the Tampa Bay Viaduct of Fig. 29, and as the deck load in this example was 50 per cent. of the live load, the variation in stress was correspondingly reduced, but was greater than the 500 lb./sq. in. quoted as not generally being exceeded by the author.

The H-20 loading used in this example appeared to be less than half of the British Standard Loading, and less than two-thirds of the H-20 loading given in a popular American text-book* on bridges, and the writer would be glad to have further details of the loading corresponding to the stresses which were given in CIVIL ENGINEERING and CONTRACTORS RECORD.

Yours, etc.,

R. G. ROBERTSON (Member).

Johannesburg.

February 27th, 1953

To the Editor of THE STRUCTURAL ENGINEER.

Sir—In answer to the questions put by Professor Robertson the occasional disintegration of a pile head is no doubt primarily due to the principal tensile stresses which develop due to uneven impact pressure, particularly where the centre of pressure is eccentric and the concrete is not sufficiently restrained by the binding at the head. They may be too briefly explained but I do not think that the varying speeds of the stress waves in the steel and the concrete are of primary importance as regards the head failures normally encountered but, of course, I am not overlooking the high head stresses disclosed in the research by Drs. Glanville, Grime and Fox.

I do agree, of course, that driving hollow piles with a mandrel has many advantages particularly for prestressed piles ; hollow prestressed piles have not yet, so far as I know, been driven that way but that is probably only because they could be driven safely and more cheaply with a special helmet.

With regard to page 306, it is of course correct as stated in the paper that the greater ultimate strength for given concrete and high tensile steel areas, is obtained by having the steel at the periphery. If it is at the centre, although it has the apparent advantage of all being usable in tension, the concrete strains on the tensile face prevent more than about $\frac{3}{4}$ the full strength of the steel being used at about half the normal lever arm.

In the design shown in Fig. 21 the slabs are bolted on to the web stiffeners and not applied to the flanges, and what appears to be struts are in fact tiles and waterproofing used to protect the bottom flanges of the girder.

As regards the variation in concrete stress due to the fluctuation of live load and my pointing out that the variation in steel stress is small and only m times 500 lb. per sq. in. in the case mentioned on page 310 the figure of 500 lb. per square inch may be a little low for an average case. However, for the three larger contracts mentioned in the paper the change in concrete stress adjacent to the lowest bar due to application of the live load was of the order of 500 lb. per square inch for the Tampa Bridge, 900 lb. per square inch for the Lee Conservancy Culvert and 1100 lb. per square inch for the bridge at Sheffield ; using this latter figure the range of steel stress is still only about $2\frac{1}{2}$ tons per square inch. Values of the order of 2,000 lb. per square inch are not usual, when the fluctuating live load only is

*L. E. Grinter, THEORY OF MODERN STEEL STRUCTURES Vol. 1.

sidered, and, of course, tensile stresses in the concrete der live load are not used in a fully prestressed ucture ; for partial prestress the considerations are ferent.

In answer to Professor Robertson's query about the mpa Bay Bridge, suggesting that the loading given a text-book differed from that quoted, may be plained by the fact that the loading used generally design was H15 but a check was made to ensure that der H20 there was no tension in the main beams. e stresses mentioned in CIVIL ENGINEERING (London) er to H15 while both were given in CIVIL ENGINEERING .S.A.) for January, 1953.

Yours, etc.,

DONOVAN H. LEE (Member).

ndon, S.W.1.
March 17th, 1953.

the Editor of THE STRUCTURAL ENGINEER.

Straub's History of Civil Engineering

Sir—Excuse the combination of a Letter with a Book eview, chosen because I wish to rouse not only interest it the discussion of a grievance shared by all who hope r an "understanding of the history of our art," thetically claimed by Professor A. G. Pugsley in HE STRUCTURAL ENGINEER, 1951, p. 14. "cremona" a noun in some Eastern languages and means his opular diagram. But do we really know much more Luigi Cremona, the late, lovable professor of Milano, of the correspondence of mutual esteem between m and Sir Thomas Hudson Beare, of Edinburgh, and arl Culmann of Zurich fame? Cremona's works, onumentally edited by the Italian Academy, are dly missing in our Institutions and, like the treatises Alberto Castigliano, or for that matter, of our own lerk Maxwell, must be traced in the British Museum.

Editors, firms, public authorities are strictly served ith author's acknowledgements : yet who but Otto ohr, Professor Fisher Cassie and a few textbook riters care to reveal the true sources of our science? ven the conscientious Dr. S. Stewart forgot to name rof. E. Winkler, the brilliant inventor of his Influence ines (1947). Worst offenders are oversea.

This is why we hailed whole-heartedly Mr. Hans traub's *History of Civil Engineering* (Basel 1949), w available in English (London, 1952). There were, course, histories of architecture, science, mathematics nd inventions and monographs on limited subjects, e Todhunter and Pearson's on the theory of elasticity, Dr. S. B. Hamilton's recent sketch on structural nalysis. But the bulk of records was dispersed in eriodicals, much of it in the JOURNAL OF THE INSTITUON OF CIVIL ENGINEERS, and in Continental collections, ostly of the past century.

Thus Mr. Straub can be congratulated on having broken almost virgin ground. The coign of vantage of this Swiss engineer residing for years in Rome was well chosen. He is familiar with art and building, well-read in our French "founder-fathers" and if he slightly favours Swiss descriptions and German authors to the detriment of English, this is neither wonder nor reproach for the pioneer work of an amateur in the noblest meaning of the word.

Perhaps his dual purpose, to reach engineers and the general reader as well, was no advantage. However, it helped us to some views on architecture, on contractors-plant, on mathematics, etc., and to the welcome refresher of selected illustrations. After all, in many matters we are general readers ourselves. There is ample documentation without the dry thoroughness of professional historians, a good—if not exhaustive—index and an interesting chronological table. Nor does Mr. Straub hide his own opinions on culture, taste and education.

Mr. E. Rockwell's translation clung too much to the original wording, quotations in foreign languages and bracketed dimensions in meter will appeal less to English readers, also a few terms of trades and of analysis sound rather foreign. Perhaps in the next edition the knighthood of Sir Benjamin Baker will be restored, even if Sir Isaac can dispense with a title.

Hoping for this event and before we shall be swamped with a less generous production of our American brethren, this book, of much educational value, should appear in all technical colleges, secondary school and public libraries and—*pace*, resident and administrative civil engineer! on your bedside table : it is the true detective story of your own trade.

Only then will the spirit of humility and of thankfulness to our betters return to our graduates and to engineers who ceased to be readers a long time ago. And then we shall be spared the foreword of an otherwise quite readable book on bridge structure (London 1930) to be pilloried as follows :

". . . but a knowledge of mathematics is of less value to the practical engineer, than a gift of expression" (salesmanship?) ". . . men can readily be hired to make calculations . . . the really difficult problems of structural engineering are not mathematical and he is able to employ mere computers for much of his work, because mathematics required are elementary and really important matters are settled by judgement and experience rather than by calculations" (*vide* failure of Tacoma Narrows).

By all means, gentlemen, let us learn history!

I am, Sir, yours very faithfully,

P. M. TEZNER (Member).

Sydney,
April 10th, 1953.

Book Reviews

Design and Placing of High Quality Concrete, by . A. Stewart. (London : Spon, 1951.) 9 in. × 5¾ in., . 112. 25s.

The burden of Mr. Stewart's book is to generate a reater appreciation of concrete as an engineering aterial. Unfortunately, the structural engineer is ill liable to give more attention to the mathematical nalyses of concrete structures than he does to the anufacture of the materials of which these structures e built. He can, of course, quite easily leave the anufacture of the steel to the metallurgist but if the concrete in a structure is to have the necessary desirable qualities such as compressive and flexural strength, then he must give some attention to its manufacture. We are gradually moving towards the time when it shall become essential, as some members of the Institution have already suggested, for all qualified structural engineers to have training in concrete technology. In this respect, I quote from the first chapter of Mr. Stewart's book.

"While it can be argued that there is little reason for including the metallurgy of steels in the curriculum of

the civil engineering student because he will never be called upon to carry out steel production on site, the same argument by no means applies to the production of concrete. Despite the fact that concrete-making is as complicated a chemical engineering process as the other, and furthermore is conducted in extremely difficult conditions of environment and labour, the student is given only the most superficial instruction in the subject. Bearing this in mind it is little short of miraculous that the combined use of concrete and steel has reached the point of efficiency it has to-day. Progress in structural design demands that a much more complete course in concrete technology should be available to students, where they may learn the elements of making and placing high-quality concrete—a material for which they themselves will be responsible in later life."

This is not a book for the superficial reader, or one that provides all the answers. It requires careful study and close attention and should provoke discussion and controversy. It places the technique of the design and manufacture of concrete on a high scientific plane, challenges present-day concepts and preaches new theories, particularly in the realm of compaction by vibration. One does not, at the first reading, agree with all that Mr. Stewart says—not by any means—but what he says is sufficient to make us think and re-examine our own ideas. In doing this Mr. Stewart has achieved something which is fundamental to a serious critical study.

One can easily criticise adversely, but as the book will be read by many who are not students or research workers, it would be useful if in a revised edition Mr. Stewart discussed the analysis of vibration in more simple terms with illustrations that even a foreman in a precast concrete factory could understand. The same remarks apply to his theory and method of mix design, which could be improved by practical examples.

One important feature completely missed from the book is the subject of curing. Granted it is not included in the title, which limits the subject to "design and placing," but Mr. Stewart will be the first to agree that all efforts in design and techniques in placing will be completely lost if the concrete is not adequately cured; without curing the strength of which the cement is capable will never be realised. This is not condemnation of the book, but a suggestion that in future editions it should be included.

True economy in cement lies in making better and stronger concrete and the vibratory method of compaction which Mr. Stewart discusses so extensively in this book, together with the other methods such as pressure and spinning, is bound to be used more and more extensively in achieving this desirable result. All serious students of concrete manufacture should read this book.

W. H. R.

Mechanics—Part I, Statics, by J. L. Meriam. (London: Chapman & Hall.) 9¼ in. × 6 in., pp. 340. 32s.

At first sight, this book appears to be of more interest to mechanical than to structural engineers because so many mechanisms are illustrated; but the theory of statics underlies all structural engineering, and this work is an original and interesting approach.

There are in effect ten sections, covering basic principles, force systems, equilibrium in 2 and 3 dimensions, the analysis of simple frames and a good discussion on the centroids of areas and volumes. There are short sections on flexible cables and hydrostatics, an introduction to shear and bending in beams, and a chapter on Virtual Work and its application to frames and linkages.

The author's object has been to develop in the reader the ability to isolate essentials in practical problems, and to tackle them logically. He does this by including a large number of problems illustrated by excellent and informative diagrams. Here are no disembodied bars poised mysteriously in space: the whole appliance or structure is shown, and the problem becomes real and lifelike. There are some 700 examples covering a surprising range of statical applications, varying from a claw-hammer drawing a nail through a variety of structures to a compound bolt-cutter less innocent than it looks. Answers are given to more than half the examples.

The text occupies less space than the problems, but it is lucid, concise and admirably written. The treatment proceeds from first principles, each section being followed by examples arranged in order of increasing difficulty. If a criticism is to be made, it is that the advance is perhaps too swift. On the other hand, some of the examples appear more difficult than they are, and this, too, is true to life.

The high standard of production is the only American note about this book, which may be commended to all engineering students and indeed to older men. The publishers describe the work as a refreshing treatment of the subject; the claim is just.

F. H. B.

Strength of Materials, by John B. Thirlwell, B.Sc.(Eng.). 206 pp., 8½ in. × 5½ in. (Macdonald, London.) Price 20s.

Although this is described as "a student's book," the qualified engineer will find it a mine of information and may be glad to turn to it again and again in the course of his work.

Here, ready to hand and very clearly stated, he will find all the theorems and principles involved in the subject of Strength of Materials, from Hooke to Castigliano, together with many of the more useful applications of those theorems and with numerous exercises added for good measure.

The preface states that "The intention is to present the basic theory of Strength of Materials with enough included to cover the whole of Part 1 B.Sc.(Eng.) for London University and sufficient additional material to make the work suitable for the Higher National Certificate candidate."

The author is to be congratulated on achieving his object with such an economy of words and having produced a book which, although employing orthodox methods almost exclusively, is never ponderous or obscure in its presentation.

For the student the value of the book is enhanced by the inclusion of an introduction to the subject of Influence Lines: the chapter on Strain Energy is particularly good, and a final chapter on The General Properties of Materials deals with such matters as Tensile Tests, Fatigue and Creep.

This book could be expanded quite easily to two or three times its present size without padding or undue digression, and for this reason the student will find it necessary to assimilate its concentrated contents in small doses.

The print and binding are good, the 110 diagrams are very clear, and for a first edition, there appear to be few printing slips. On page 72, 21 appears as 21 and $\log_{10}\triangle$ in Fig. 110 should be $\log_{10} A$. Also, owing to their similarity, the use of the symbols δ and ∂ is rather confusing in some parts of the book.

C. A. R. E.

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of ructural Engineers was held at 11, Upper Belgrave reet, London, S.W.1, on Thursday, June 25th, 1953, 5 p.m. Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., .I.Struct.E. (President), in the Chair.

The following members were elected in accordance ith the Bye-Laws. Will members kindly note that ıe elections as tabulated below should be referred to hen consulting the Year Book for evidence of member-ıip ?

Students

LDRED, Donald Herbert, of Epsom, Surrey.
RADLEY, Denis, of Wembley, Middlesex.
ROWN, Francis Stanley, of Scunthorpe, Lincs.
ARTER, Clifford, of Salford.
HESTERTON, Charles John, of London.
HIN KEE KHEONG, of Brighton.
RITCHLEY, Leslie Barker, of Swinton, Lancs.
EESON, Thomas Godfrey, of Port Elizabeth, South Africa.
IXON, Gerard, of Manchester.
ULSON, William, of Salford.
RANCIS, Rhys Hugh, of London.
IBBONS, John Frederick Pearson, of Aston-on-Trent, Nr. Derby.
RUNDY, Frank Stephen, of Swinton, Lancs.
ARRIS, George Graham, of London.
OWES, Roy, of Bolton.
UGHES, Robert Edward, of Saughall, Cheshire.
UNTER, Kenneth, of Hayes, Middlesex.
OURSOUMBAS, Phidias Georghou, of London.
EE TECK CHIOW, of Brighton.
IDDLEMISS, Joseph Watson, of Newcastle-upon-Tyne.
ORGAN-GREEN, Peter, of London.
ORRIS, Anthony, of Upton-by-Chester, Cheshire.
ERRY, Ronald Edward, of London.
EYNOLDS, David Edward Holden, of Scunthorpe, Lincs.
METHURST, Gordon James, of Bolton.
PEIGHT, Peter Alfred, of Leeds.
HOMASON, William Brian, of Esher, Surrey.
EDLAKE, David Harry, of Cyprus, M.E.L.F.
ELLINGS, Paul Eustace, of Bishops Stortford, Herts.
ILLINGTON, John Ernest, of Manchester.
OOD, George Alan, of Gateshead.

Graduates

AILEY, Frank, of Salford.
ARNWELL, Derek, of Audenshaw, Lancs.
EVAN, Iorwerth, of Port Talbot.
HANDARI, Dali Ardeshir, B.E.(Civil) Poona, of Bombay.
HIDE, Vitthal Sadashil, B.E.(Civil) Poona, of Poona.
LAPHAM, Harold George, B.Sc.(Hons.) Wales, of Pontypool, Mon.
LARKE, Dennis, B.Sc.(Civil) Durham, of Newcastle-upon-Tyne.
OLLIER, Ian, of Stretford, Nr. Manchester.
AY, John Gordon, B.E.(Civil) Queensland, of Brisbane, Australia.
EAR, George Cornelius, B.Sc.(Eng.) London, of Portishead, Somerset.
EMPSEY, Bernard, of Sale, Cheshire.
LLIOTT, Hubert Edward, B.Sc.(Hons.) Swansea, of Port Talbot.
FAIRLEY, Archibald Thompson, B.Sc.(Eng.) Edinburgh, of Wakefield.
FOSTER, Denys Willshaw, of Manchester.
GEESON, David Arthur, of London.
HAM, Roderick Thomas Mathieson, A.R.I.B.A., of London.
HARRISON, David Baldwin, B.Sc.(Eng.) Rand, of London.
HEATH, Roy Edward, of London.
HUGHES, Gareth, of Llanelly.
JOHNSON, Geoffrey Philip, of Banbury, Oxford.
JUDAS, Josef Erwin, of Birmingham.
KELLY, Charles Joseph, of Manchester.
KENNEDY, Christopher David, of London.
KRAWCZYK, Eugeniusz, of London.
LUKER, Colin Rae, of London.
MACKENZIE, Roy Macgregor, of Edinburgh.
MILLS, Ralph Lawson, of London.
MOORE, John Denis, of London.
MORAN, Thomas Francis, of Liverpool.
MORGAN, Robert Evan, of Bridgend.
MURPHY, Thomas Philip, of Rusholme, Manchester.
MURRAY, Stewart Forbes, of Stoneleigh, Surrey.
PATRICK, John George, B.Sc.(Eng.) London, of Banstead, Surrey.
PHALP, Brian Trevor, B.A.(Hons.) Cambridge, of Keighley, Yorks.
PRITCHARD, Jonathan, B.A.(Hons.) Cambridge, of London.
PROSSER, William Henry, of Bridgend.
RADCLIFFE, George Frederick, of London.
RYDER, Donald Edgar, of Manchester.
SCHUBERT, Sydney, B.E.(Civil) Queensland, of Queensland, Australia.
SIMMONDS, Stanley Bryan, of Birmingham.
SIMMONS, Maurice Arthur, of Birmingham.
STEWART, Peter Henry, of Loughborough.
SUTCLIFFE, Roy Holtham, of Bolton.
TAGUE, Lancelot James, of Wallasey, Cheshire.
WHITE, John Davis, of Aldermaston, Nr. Reading.

Associate-Members

AHLUWALIA, Devi Chand, of Punjab, India.
DUNCAN, John, B.Sc.(Civil) Glasgow, of Glasgow.
ROBINSON, Thomas Algie, A.M.I.C.E., of Dunfermline.

Members

JORDAN, Sydney Kenneth, A.M.I.C.E., of Claremont, Cape.

Transfers

Students to Graduates

CLARK, Tony Allen, of Chigwell, Essex.
DAVIES, Colin, B.Sc.(Eng.) London, of Treorchy, Rhondda.
LANE, Dennis John, of London.
LEADBEATER, Geoffrey, of Knutsford, Cheshire.
WILKES, Brian Edward, of Worcester.

Graduates to Associate Members

BARTLE, Peter Ronald, of London.
BURR, Robert William, of London.
DHAVALIKAR, Dattatray Ganesh, B.E.(Civil), of Ahmedabad, India.

DIVERALL, William, B.Sc.(Hons.) (Civil) Glasgow, of Glasgow.
FINCHAM, John Gordon, of Sutton, Surrey.
FINN, Edward Vivian, of London.
HALE, Brian Thomas, of Fremantle, W. Australia.
JOLLY, Roy Cecil, B.Sc.(Eng.) London, of Nottingham.
KIMBER, Basil Richard, of Woodford Green, Essex.
LONGBOTTOM, Derek Patterson, B.Sc.(Civil) Durham, of Bradford.
SUBBA RAO, Tippur Narayana Rao, B.E.(Civil), of Bombay.
SUNDLO, Nils Asbjorn, of Norway.
TRANFIELD, William, of Wembley, Middlesex.
WOODBURN, George, B.Sc.(Hons.) Edinburgh, of Vancouver, Canada.

Associate Members to Members

BARTLETT, William Harry, of Nakaru, Kenya.
HOOKER, Arthur Vernon Robert, A.M.I.C.E., of Port Talbot, Glam.
PINK, Ronald Norris, M.B.E., of Mitcham Park, Surrey.
SLY, Victor Stanley, B.Sc.(Civil), A.M.I.C.E. Birmingham, of Kuala Lumpur, Malaya.

Member to Retired Member

MARSHALL, Allan Edward, of Ashford, Kent.

OBITUARY

The Council regrets to announce the death of JOHN DUNLOP ANDERSON, DAVID WILLIAM DITCHBURN, PAUL THORZ, THOMAS WALLIS, ERIC GEORGE WILLEY (Members) ; THEODORE DELABERE BARKER (Retired Member) ; HERBERT WALKER HIGSON, ERNEST JONES (Associates) ; VICTOR THOMAS LOW, GEORGE FREDERICK POPPLETON, FRANZ HERMAN PREIJER, ELLIOTT REEVE, EMMERICH ROSENBAUM, ERNEST VICKERS (Associate-Members).

MR. J. DUNLOP ANDERSON

Mr. J. Dunlop Anderson, O.B.E., J.P., M.A., B.Sc., M.I.Struct.E., who died on February 26th at the age of 71, was a former member of the Council. He had been a member of the Institution since 1931 and was a Past Chairman of the Scottish Branch, in the affairs of which he took the keenest interest. Mr. Anderson was born and educated in Glasgow. After graduating in 1908, he taught for a time before deciding to make engineering his career. He then became an apprentice with Alexander Findlay & Co., Ltd., of Motherwell. His professional work included the construction of bridges and dams in Egypt and the Sudan. In addition to engineering, Mr. Anderson continued his interest in education, particularly in connection with technical and University education in Glasgow, and in October, 1946, he was elected an Assessor of the General Council on the University Court.

He was a man of wide interests, friendly and forthright, and his loss is deeply felt in Scottish engineering and educational circles.

MR. SCOTT H. HUME

Mr. Scott Hamilton Hume, M.I.Struct.E., who died suddenly on December 24th, in his sixty-second year, had been a member of the Institution since 1924. He was awarded the Institution Bronze Medal for a paper on site organisation which he read during the Session 1942-43.

MR. H. A. WHITAKER

Mr. H. A. Whitaker, M.Eng., A.M.I.C.E., who died on January 26th, was 61. He was elected a Member of the Institution in 1923, and was a Past Chairman of the Yorkshire Branch and of the Lancashire and Cheshire Branch. He was awarded the Institution Silver Medal for a paper on the mechanical solution of influence diagrams, read before the Lancashire and Cheshire Branch during the Session 1931-32.

RESIGNATION

Notification was given that the Council had accepted with regret the resignation of CHARLES HAROLD HEATHCOTE (Retired Member).

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, October 1st, 1953

Presidential Address, by Lt.-Col. R. F. Galbraith, M.C., R.E., B.Sc., M.I.Struct.E., A.M.I.C.E. 6 p.m.

Thursday, October 22nd, 1953

Ordinary General Meeting, for the election of members, 5.55 p.m., followed by a Joint Meeting at 6 p.m., with the British Iron and Steel Research Association and the Engineers' Group of the Iron and Steel Institute, when Mr. J. S. Terrington, B.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E., and Mr. J. M. Hawkes, B.Sc., Ph.D., A.M.I.C.E., A.M.I.Struct.E., will give a paper on "Crane Gantry Girders for Steelworks."

EXAMINATIONS—JANUARY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 5th and 6th (Graduateship), and January 7th and 8th, 1954 (Associate Membership).

REPRESENTATION

The Council has made the following nominations of members to represent the Institution :—

CITY OF NOTTINGHAM EDUCATION COMMITTEE, ADVISORY COMMITTEE :
Mr. D. G. Milloy (Associate-Member).

IMPERIAL COLLEGE OF SCIENCE AND TECHNOLOGY—Concrete Technical Advisory Committee :
Mr. Ernest Granter (President)—reappointment.

HONOURS AND AWARDS

In offering their sincere congratulations to the following member on the distinction recently conferred upon him, the Council feel that they are also expressing the good wishes of the Institution :

ORDER OF THE BRITISH EMPIRE—M.B.E.
Mr. A. E. Cotton (Member).

ENGINEERING, MARINE AND WELDING EXHIBITION

The Directors of the Engineering, Marine and Welding Exhibition, which will be held at Olympia from September 3rd to 17th, 1953, have invited all members of the Institution to visit the Exhibition on Tuesday, September 15th, 1953. Tickets of admission, each available for two persons, which may be used for the official visit or on any other single day, may be obtained on application to the Secretary.

COMMUNICATIONS TO MEMBERS

The Institution is experiencing difficulty in receiving replies to communications addressed to the undernote

iembers, and the Secretary would be glad to be notified f their present addresses :—

R. S. Thompson	Member.
W. S. Blount	Associate-Member.
G. Connelly, N. C. Sagar	Graduates.
J. X. Lewis	Student.

PAPERS FOR PUBLICATION

The Literature Committee would be glad to consider ffers of papers for presentation at the Institution or or publication in the Journal.

The following is a summary of the Committee's equirements relating to articles and papers : a copy f the full conditions may be obtained from the Secretary.

(1) Articles must be of an appropriate character, aving a bearing upon structural engineering or upon ome kindred scientific or constructional subject, and nust be approved by the Literature Committee. A hort title is an advantage.

(2) Contributions must be original either in subject-natter or in presentation. Articles which have already been published or have been read to other organised odies, or are carelessly prepared, will not be accepted or publication.

(3) The style of writing will necessarily vary with the ndividual, but authors are requested to write as plainly and simply as their subject will allow. Papers should e written in the third person.

(4) Where the subject allows, a brief introduction or ynopsis should state clearly the purpose and scope of he paper or article, and the author's conclusions or ecommendations should be summarised at the end of he paper.

In order to facilitate the indexing of articles for eference, the author will be required in addition to prepare a short precis not exceeding 25 words for inclusion under the title of the paper on the contents page f the Journal.

(5) Illustrations are desirable where they assist in explaining the context or are fundamental to the ubject. They should not be used if unnecessary for hese purposes. Illustrations may be either line drawings or photographs.

(6) Line drawings must be specially prepared for eproduction on smooth white paper or clear tracing aper, with heavy main lines and large clear lettering rawn in Indian ink with a mapping pen. Alternatively, he author may submit drawings on one sheet of paper ith the relevant lettering on a cover sheet of tracing aper.

The printed page of THE STRUCTURAL ENGINEER is in. wide by 10 in. deep. The drawings, where practicable, should be prepared not larger than twice this ize with a view to half-scale reproduction. Unavoidably arge drawings which require reduction to one-third size r less, must be specially heavy and with proportionately arge lettering for clear reproduction. Ordinary working rawings are not satisfactory.

(7) Where photographs are submitted they should e printed *black on glossy paper*.

(8) MS. typewritten in double spacing should be ubmitted in duplicate.

Brevity is an advantage and papers should not ormally exceed 7,500 words in length.

DRURY MEDAL AWARD

The fourth competition for the above award will take lace in 1953. The subject is the design of the structure f a new factory building. The material of construction s entirely at the choice of the competitor. The competition has been designed to encourage ingenuity of structural arrangement. Economy in the use of steel is an important feature of this year's competition.

Graduates and Students of the Institution who wish to compete are invited to apply for full details to the Secretary ; envelopes to be marked in the top left-hand corner, " Drury Medal Award."

The closing date for the competition is October 1st, 1953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution or, failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

RESEARCH AWARDS

The Council has instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of construction ;
(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;
(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the science and art of structural engineering ;

(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1952, and September, 1953, is October 1st, 1953.

LONDON GRADUATES' AND STUDENTS' SECTION

By courtesy of Messrs. Braithwaite & Co., Engineers, Ltd., a visit has been arranged to their Newport Works on September 3rd and 4th, 1953.

The party will leave Paddington Station at 6.45 p.m., on Thursday, September 3rd, and arrive in Newport at about 9.45 p.m., and then proceed to hotel accommodation which will be arranged in advance. On Friday morning, the party will be taken to the works to see the layout and arrangement of Fabrication Bays for Welded and Riveted Structures in steel, plate pressing bays, and aluminium welding sheds.

In the afternoon the party will visit building site in the locality to see the work in progress. It is hoped to leave Newport by the 6.40 p.m. train, arriving in Paddington at 9.25 p.m.

All transport, lunch, tea, etc., in Newport will be provided by the company.

The cost of accommodation and fares is not expected to exceed £3, and all members wishing to attend should submit their names to the Hon. Secretary as soon as possible. The closing date for bookings is Friday, August 21st, 1953.

A special welcome is extended to those members of the Midland Counties Branch, who wish to attend.

Hon. Secretary: Mr. J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

CONFERENCE ON WELDED STRUCTURES

A conference will be held in London on November 23rd-27th, 1953, to review present knowledge and outstanding problems in the design, fabrication and erection of welded structures.

The organisation of the conference is being undertaken by a joint committee appointed by the Minister of Works, the Institution of Civil Engineers and the Institution of Structural Engineers with the support of the Admiralty, the War Office, the Ministry of Transport and the Department of Scientific and Industrial Research.

Working sessions will be held at the Institution of Civil Engineers, Great George Street, London, S.W.

Further particulars will be published in the September issue of THE STRUCTURAL ENGINEER, but readers desiring earlier information regarding the conference are advised to communicate with the Secretary.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

Hon. Secretary: A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

MIDLAND COUNTIES BRANCH

Hon. Secretary: L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

MIDLAND COUNTIES BRANCH—GRADUATES' AND STUDENTS' SECTION

Hon. Secretary: F. G. Fletcher, 60, Brean Avenue, South Yardley, Birmingham, 26.

NORTHERN COUNTIES' BRANCH

Hon. Secretary: O. Lithgow, A.M.I.Struct.E., 4 Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

Hon. Secretary: S. G. Duckworth, M.I.Struct.E., "Lisleen," 13, Finaghy Road North, Belfast.

SCOTTISH BRANCH

Hon. Secretary: G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The Annual General Meeting of the South-Western Counties Branch was held at the Duke of Cornwall Hotel, Millbay, Plymouth, on Friday, May 22nd, 1953, at 6.30 p.m., when the following honorary officers and Committee members were elected for the Session 1953-54 :—

CHAIRMAN :	Col. R. Hazzledine (Member).
VICE-CHAIRMAN :	E. W. Howells (Member).
JOINT HON. SECRETARIES :	E. W. Howells, 10/12, Market Street, Torquay, Devon.
	C. J. Woodrow (Graduate).
HON. TREASURER :	E. W. Howells (Member).
HON. AUDITORS :	H. J. Scoles (Member).
	J. C. Peters (Associate).
COMMITTEE :	L. F. Vanstone (Member).
	F. J. Powell (Associate-Member).
	E. W. Howells (Member).
	C. J. Woodrow (Graduate).
	J. C. Peters (Associate).
	H. J. Scoles (Member).
	F. J. Dean (Associate-Member).
	A. C. H. Harris (Associate-Member).
	F. M. Upson (Associate).
	W. C. Tyler (Associate-Member).
	H. Toft (Associate-Member).
	F. W. Potter (Associate-Member).
	H. W. Miller (Graduate).
	Col. R. Hazzledine (Member).

SCIENCE COMMITTEE DELEGATE : Col. R. Hazzledine (Member).

BRANCH REPRESENTATIVE ON THE COMMITTEE OF MANAGEMENT OF THE BENEVOLENT FUND : H. J. Scoles (Member).

The Branch Programme for the ensuing year was discussed at length and various suggestions adopted.

WALES AND MONMOUTHSHIRE BRANCH

Hon. Secretary: G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 2 Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

Hon. Secretary: E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

Hon. Secretary: E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary: A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111. Ext. 257.

Natal Section, Hon. Secretary: E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary: R. Stubbs, M.I.Struct.E. P.O. Box 1692, Cape Town.

Economy in the Use of Structural Materials*

By G. A. Gardner, O.B.E., M.I.Struct.E., R. Morton, B.Eng., M.I.C.E. and L. R. Creasy, B.Sc., M.I.C.E.

GENERAL

By G. A. Gardner

The authors have been asked to write this Paper by ay of expounding and illustrating some of the matters ised in the Report issued by the Heads of the Works irectorates of the Ministry of Works and Service epartments on the Economic use of Building Materials, nd they are limiting their observations to those subjects more particular interest to the members of this nstitution, and are including reference to the Steel conomy Bulletins.

The decision to present such a Paper was made to give ffect to the desire expressed in the general introduction the Report that the Architectural and Engineering nstitutions might co-operate in propagating the basic leas of economy, with a view to their wide acceptance y all those concerned with construction; and the nstitution volunteered to give publicity to such a aper. It might, at first sight, seem strange that public ervants whose observations had been embodied in the eport should be selected to give effect to this, since they, eing practising engineers, moulded to the current attern, have *ipso facto*, nothing new technically to resent of which the profession, as a whole, is not aware, nd any items which might transcend the experience of nore junior members are amenable to investigation long the lines suggested. However, the Council of the nstitution deemed this the most helpful course to follow nd it is hoped that it will be useful in reviewing some of ne questions involved. By way of illustrating some of ne broad precepts of economy we may observe that where, or example, it is said that economy in steel in the case of ne normal articulated roof truss is effected by keeping pans small, it is a fundamental fact of static mechanics nd is displayed as a reminder to those who may select arge spans for reasons which have nothing to do with teel economy. Likewise, when it is said that a welded russ is lighter than a riveted one, and yet again that a ubular truss is lighter still, it is also a fact and a re- inder to those who may select riveted work for econ- mic reasons of quite another kind. I, personally, am ully persuaded that the only fundamental economy is hat of human energy in its relationship to utilizing naterials and using locked-up energy, and that in con- equence the sum total of human achievement, whether vorthwhile or not, is proportional to the care with which uman energy is used. If this is so, true economy in any iece of construction must be related to the total amount f human energy needed for the conception, building and naintenance of the structure, and this is, of course, not ecessarily commensurate with the use of a minimum mount of *material* in any particular case. We may have simple steel roof truss of hot-rolled members which uses ess human energy in the above respect than a lighter and nore complicated truss, and so we see that the concept f material saving alone is a loose one which only becomes imperative when the particular material under consideration is in short supply, and when, in consequence, we must re-adjust the incidence of the applied human energy—more brain power and less manual force, for example. Furthermore, when we take into account, as we must do, that complicated factor cost, as ordinarily understood, i.e., the amount of money which is exchanged in the process of getting what we want, the issue becomes further obscured and the fundamental concept which I have cited is lost in a mass of inter-related conditions and processes.

All this being so however, when we speak of Economy in the use of Stfuctural Materials we nevertheless voice a broad concept which seems sensible by comparison, *per contra;* that is we are all persuaded that to advocate ignoring such a broad concept is irrational at the present time, and that we all know in a general sense what is in view.

My colleagues in this Paper will be expanding this fundamental concept with typical exemplifications in several types of construction, in accordance with the Institution's request, while I propose further to examine a few of the general aspects of the matter.

Reference has been made in the Report to the considerable effect that the initial concept and detailed design can have on the economic use of materials, and this, which is a self-evident truth, is, nevertheless, not infrequently overlooked. The broad principle mentioned in the opening remarks concerning the effect of span dimensions is a very important one but, as everybody knows, it must be weighed with considerations regarding foundation design; and when the Report makes a plea in this respect it is with this very point in view, namely that the major structural economy should be the operative motive, all things considered. Small spans are perhaps a little old fashioned and neither they, nor the common roof truss, are liked by the Architect with modern tendencies, but the fact remains that such construction is very difficult to beat economically in a great many cases. The graph in Fig. 1 shows this effect of span on the weight of steel used per 100 square foot of floor area covered, and also indicates the economic relationship between the several types of construction, such as riveted and bolted or welded, all designed normally to B.S.449; and so we could go on establishing such data for numerous types of roofs and structures generally. With the aid of such graphs as these, an early decision can be made in the sketch plan stage and the dictates of economy can be duly weighed.

The anatomy of structures tends to be rather neglected in the general rush of business as a conscious approach to basic economic design and a lot of useful work could be done to establish, for practical use, the trends in this direction; and it may well be that we are harping too much on stress limits and not enough on anatomy. Numerous elementary instances occur such as the following :—

A factory is required to be of two 60 foot bays with north light trusses, and the Architect chooses a short span for the trusses to satisfy aesthetic consideration

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1. on Thursday 16th April 1953. Mr. E. Granter, B.Sc., M.I.C.E., M.I.Struct.E., (President) in the Chair.

externally. The 60 foot spans are required to carry a central runway capable of lifting five tons, and the economic index is at once seen to be the spacing of these girders, since they must all be subjected to the runway load.

taking into account the super-added vierendeel acti which must develop on initial deformation and su things as utilising the counter action of pre-arrang eccentric connections. A good example of the latter w brought to my notice by the Dutch Engineer, M

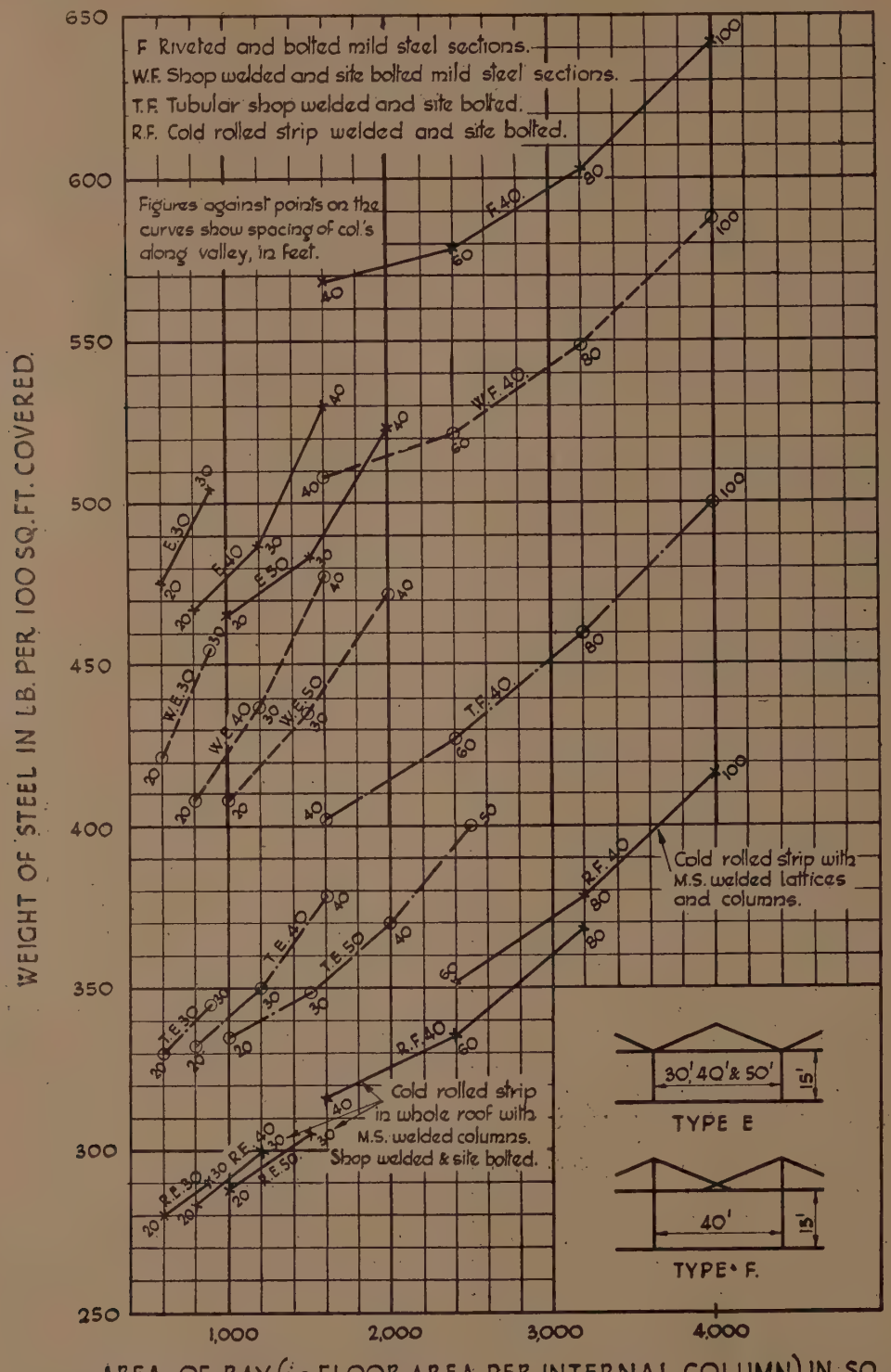


Fig. 1—Simple pitched roofs

In this case the aesthetics should, perhaps, bow to Structural Economy, not that I am suggesting that structural economy is necessarily inimical to aesthetics. And then the basic analytical approach in design can be fruitful of material savings. I think it would pay to review our graphic statics approach in important cases and establish correction factors where appropriate,

van der Eb. One hundred and twenty 16-metre sp trussed deck Railway bridges were required for t replacements. They were each composed of t Warren girders with a single track carried by sleep supported directly from the top cords, and advantage w taken of the moment resulting from the horizontal co ponent of the Warren system being arranged in the pl

the bottom flange of the top cord as a counterbalance the local bending moment of the rolling load from a o-ton Mallet locomotive. This feature which can early be seen in Fig. 2 gives an obvious mechanical vantage apart from the strain energy basis which was opted analytically in designing these bridges and which ogether saved a considerable tonnage of steel. This inciple of counterbalance due to eccentric connections, not, of course, new, and has been utilised in the design ordinary roof truss rafters, but it is not common and ems to be worthy of more thorough investigation where petition work is involved. I am fully alive to the fficulty of finding time to bring these refinements into e workaday drawing office, and that in order to make

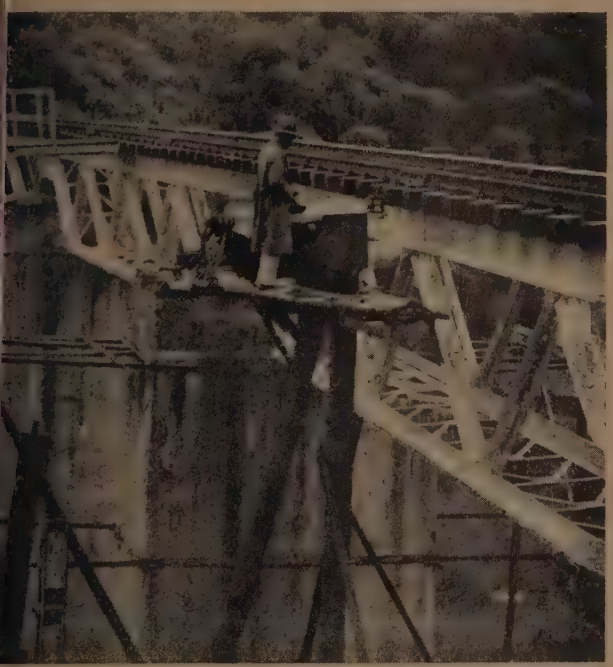


Fig. 2

em a commercial proposition somebody has got to mplify procedure; but as Ruskin said "I am here ncerned primarily with the fundamental truth of rinciples and not with their immediate effect."

The utilising of walling in conjunction with framing is nother matter which is in direct line with the question of aterial economy, and this can range from the load-earing wall to the composite action of wall panels with e steel or reinforced concrete skeleton.

The load-bearing wall for structures other than mestic houses and flats has been dubbed reactionary t there is no *prima facie* case against it, and if we take to account the question of maintenance it may show an verall advantage. The Structural Engineer is fully volved in this, for the time has gone when the brick or asonry wall was almost entirely a matter of rule of umb based on traditional models, and through the ork of the Building Research Station we now have data hich can be used scientifically. But the complicated atter of composite action necessitates a more guarded proach. The utilisation of wall panels as bracing is a latively simple case and the panels can be quite easily lculated for diaphragm action in conjunction with the rtical and horizontal members of the framing (not for-tting, of course, the possible need for temporary rection-bracing) and not only may this save material nd obviate labour in cutting brickwork to accommodate racing, but also contribute to the amenities of the fac-ry. The much more complex business of the relief which wall panels may give to beams supporting them is in quite another category and is, as you know, being at present made the subject of a long-term investigation by the Building Research Station. The general principle is obvious and simple enough, namely that the panel and the beam under it tend to act as a composite member in flexure, having regard to aid from connections at the ends of the beam, etc. From permanent strain gauge readings which are being taken in one of our normal steel framed buildings the effect of long-term deformation is being noted, but we are all fully alive to the fact that the contingencies of fire, vibration and alterations following on a change of user are practical matters which must be weighed against the basic idea, and that it may well be that the scientific investigator will not be able to carry us with him completely. However, the general principle of composite action is assuming more and more importance and is something which we are all often forced to take into account willy nilly, when giving an analytical reason for the fact that an old structure stops up. The fundamental idea of economy which I previously hinted at, namely that of human energy, must however be borne steadily in mind, for even the Engineer must preserve a small factor of safety on his working expenses.

And so we could go on through the whole gamut of the points at issue and perhaps arrive at a broad consensus of opinion which, if it were made to satisfy *all* interests would, indeed, get a long way from the limited conception of mere material economy. But I return to my earlier remark, that we must all deem the broad issue one of considerable importance at the present time, and, I am afraid, for many years to come. The Structural Engineer is still subjected to an occasional gibe from the Architect and the Administrator that his structures are "too strong" and this is tantamount to an accusation that he is wasting the country's wealth; but those of us who have spent our life at this business know that we have now reached a stage beyond which every step must be taken with extreme caution and that the optimum limit is not far off.

Before I hand on the torch to my two colleagues it will not be irrelevant to recall one important general consideration mentioned in the Report, namely that of the Administrator allowing insufficient time between the initial decision to proceed with a project and the date for letting the contract. This is, in so many cases, inimical to material economy, and like the Printer's Devil in another connection, is something which is a perpetual nuisance and a great hindrance to the very thing we are striving after. Refinements cannot be risked in such circumstances and I am sure we could all bring dozens of instances forward. But presumably this particular Devil will continue to beset us and constitute a condition with which we must cope by employing rapid methods of design—a matter which I will touch on in conclusion.

The continual progress of scientific investigation into our numerous structural problems is directly connected with our reaching after an optimum material economy but its practical translation into workaday drawing office design methods is something which needs attending to more systematically. In this connection I think it is important to preserve the idea of principles, rather than mere rules but at the same time to avoid the first-principle analysis in the all but the unusual cases. What in my view is needed, is not so much the further prosecution of ultimate scientific investigation as that of a comprehensive series of tests on members and, indeed, complete structures, contrived to check our common

design methods and suggest correction factors. There is no time to pursue this here, and it is a little beyond the scope of our present review, so I will now make way for Mr. Morton and Mr. Creasy.

MARITIME STRUCTURES

By R. Morton

The application of economy measures to any works project calls for careful consideration and discretion and this applies particularly to heavy civil engineering works, since any misplaced economy in these may result in failure with catastrophic consequences, and making good may involve complete replacement of the original structure; further, periodical repairs and maintenance may be a practical impossibility and therefore the original structure must be so proportioned as to ensure as far as possible the length of life required.

It is in the general design stage of the scheme that lies the greatest opportunity for economy, not so much in paring down the size of members and reduction in quality of materials, and if the true interests of economy are to be safeguarded it may on occasion be necessary to include in the design, measures, which at first sight may appear to be if not extravagant, at least not economical.

The object of this paper is not to tell engineers what they should or should not do to effect the greatest economy in the construction of the works entrusted to them, as every engineer should be well aware of his responsibility in this direction, but rather to remind them of lines of thought or action which might otherwise have been overlooked when considering the best scheme to meet their client's requirements, and at the same time to comply with the national need for economy of effort, the latter expression covering both manpower employed on the site and that employed in the production of materials to be incorporated in the works. It is obviously quite impossible to be dogmatic on this subject since the circumstances differ from job to job and the design and execution of the works must vary in accordance with conditions imposed.

The following notes deal with some aspects of the application of these principles to the design of maritime structures.

Jetties

In the first place it must be borne in mind that jetties are usually required in the most inconvenient places from the engineering aspect. They are built over water possibly tidal and exposed to the action of waves and currents and frequently at sites where the foreshore and sea-bed is soft mud. Protection from the weather of the workmen and plant employed is usually impracticable. It is therefore, obvious that any reduction or simplification of the operations to be carried out *in-situ* is a step in the right direction. The trend of jetty design has in fact in most respects followed this line. In the days of timber jetties the piles were usually given support to as low a level as practicable by an intricate system of walings and crossed diagonal bracings, and when reinforced concrete began to replace timber as the basic material for jetties the arrangement of members was in many cases a replica of that in the timber jetties the reinforced concrete was to replace. The shuttering for the cast *in-situ* bracings and walings was very expensive and the fixing of reinforcement and placing of concrete was similarly an expensive item and the consolidation of the concrete was difficult to ensure particularly as the bracings and walings were normally carried down to within one or two feet of low water involving tidal work in the lower portions of the superstructure.

As a later development, raking piles were used allow the elimination of much of the bracing. The d however, would be formed of main beams, crossbea and decking, again calling for complicated formwork a reinforcement. The further development therefore, been to reduce and simplify the deck construction, a design of many jetties now comprises piles, vertical a raking, surmounted by a reinforced deck slab of a t form thickness of perhaps two feet, in which the heads the piles are set and from which all beams have b eliminated. The cost of the extra concrete incorpora in the thick deck slab is more than compensated for the simplifaction of the formwork and the ease of layi the reinforcement and the concrete. A further point that except where heavy crane loadings are to be p vided for, there may be no need to impose restrictions loading to the extent necessary where the deck is co posed of a comparatively thin slab and beams. T extra weight of the deck assists in neutralizing a tendency to uplift due to the use of raking piles.

The growing use of prestressed concrete in the ci engineering world is having its effect on jetty work. C very appreciable advantage of prestressed concrete wh used in structures exposed to the elements and par cularly to sea water and salt air is that if in the course the constructional operations cracks should form in t member due to temporary overstressing, when once t excess stress is relieved, provided of course that t temporary over-stressing was not too severe, the crac will close again and the damp and salt which in the no prestressed pile would penetrate to and attack the re forcement will be excluded.

Occasions will arise when thick slab constructi would not be suitable to the circumstances and a de provided with beams is required. In such a case it m be found that prestressed precast beams can be econon cally employed.

Prestressed piles may also be used with advantage f the reasons above-mentioned. Further, since t strength required for the movement of the pile from t casting ground to the pile frame not infrequent governs the cross section of the pile, the greater mome of resistance of a prestressed pile than that of a norm pile of similar cross section may permit a smaller cro section to be adopted. The advantage possessed by t prestressed pile in this respect may, however, be part neutralized by the fact that in certain strata it may found that a greater length of the smaller section p stressed pile has to be driven to achieve the required s

Jetties in common with other structures affo examples of how economy at the constructional sta may well prove to be the reverse when the long term vie is taken. In the early days of reinforced concrete cover of $\frac{3}{4}$ in. was commonly specified for the rei forcing bars with the result that after a very few yea the reinforcement corroded and caused the concrete spall off. The making good of such a state of affairs some cases proved to be a long and expensive operatio

The fendering of jetties forms an expensive part of t structure which may account for perhaps 25 per cent. the whole cost of the work. When jetties were made timber the resilience of the bearing piles was such th timbers bolted solidly to the structure provided satisfactory form of fendering both from the view poi of the jetty and the ship coming alongside. Th development of rigid reinforced concrete jetties howeve put fenders in a different light and very considerab thought and ingenuity has been devoted to their desi with a view to absorbing the kinetic energy of shi berthing, and thus reducing the effect of impact on bo

p and jetty. The cost of the various types of fenders ich have been introduced in recent years varies very nsiderably, but study of the problems involved and the option of the fendering best suited to the conditions ll be a dividend-paying investment as regards future aintenance.

In certain circumstances steel piles either built up m sheet pile sections or rolled steel joists have been ed in preference to reinforced concrete in view of their silience, resistance to bending stress, comparative htness and consequent ease of handling and transport. here, as in mooring and guide dolphins, resilience is an sential quality, structures formed of steel box piles rmounted by a thick concrete slab and devoid of acing or raking piles will often be found to meet the quirement. Similarly in jetties which are not to be uipped with cranes, a horizontal deflection of two or ree inches when a ship comes alongside may be ceptable, and in such cases a thick reinforced concrete ab carried on vertical piles with high bending resistance g. steel or prestressed concrete) will suffice without any rm of bracing or raking piles. In assessing the fective length of such piles it should be noted that the teral support provided by even the poorest mud is fficient to justify assuming the point of fixity as being bout five feet below the surface of the sea-bed, provided here is adequate penetration further into the mud eyond this point or into better material.

In calculating the total load to be carried by any one etty pile, regard should be paid to the fact that maxium loading conditions may in many cases only occur ery occasionally and the combinations of the various dverse conditions may be likely to occur so rarely as to llow them to be almost ignored. It is also the case that ome wheel loadings given by crane manufacturers arise rom assumptions which while it may well be prudent to pply when considering certain features of the crane tself (e.g. the jib) may be unreasonable to apply when onsidering the loading on each individual pile in the etty. It is not suggested that these possible maximum pper limits should be ignored completely, but if care is xercised the effects of some can be regarded as occasional ncroachments on the normal factor of safety.

Dry Docks, Slipways, Wharf Walls, Breakwaters

Many of the general principles governing economical lesign and construction referred to under the heading of etties apply to these and most other maritime structures nd equally, the necessity for care and discretion in ffecting economies applies in these cases if heavy maintenance costs are to be avoided. The following notes are ntended to bring to notice points which might be nvestigated in regard to each of the particular classes of tructure above mentioned.

Dry Docks

The Institution of Civil Engineers has recently published a memorandum on the construction of dry docks and a perusal of this will no doubt, suggest possible economies in the construction of the projected dock. The many operations involved in dock construction most of which are subject to the circumstances pertaining to the particular work in view, render it impossible to give any detailed guidance in the present paper. It may, however, be emphasised that adequate site investigation is a *sine qua non* to economical design and any tendency to economise on borings and other preliminaries may well prove to be a very false economy indeed.

It has for instance, been suggested that sheet piling might, with advantage, be substituted for the traditional heavy masonry walls of a dry dock. Before such a design were adopted however, it would be necessary to ensure that the long sheet piles could be freely driven to the required depth since any obstruction to penetration might result in heavy expense and very considerable delay in execution.

Slipways

The design of a slipway is obviously dependent on the site conditions and on the maximum weight and dimensions of the vessels to be slipped, all of which the engineer has usually no option but to accept.

The general principles of economical design will be applicable and the method of construction to be adopted should be given the most careful consideration since it may well prove that although at first sight the construction of a coffer dam to permit work in the dry, may appear an unwarrantable expense, this may in the end be the most economical arrangement and further, there is little doubt that it would produce the better quality structure below water level. If there is any requirement present or future for slipping two vessels simultaneously, the design should provide for the construction or later addition of a traverser serving multiple laying-up berths.

Wharf Walls

In years gone by, heavy masonry walls were the normal form of construction for wharves but depending on the use for which the wharf is intended, site conditions, and other factors, steel or reinforced concrete (preferably prestressed) sheet piling tied back to anchorages will frequently serve the purpose and the deep excavations, often in most unfriendly strata, may thus be avoided, with of course, the saving of the large quantities of concrete or masonry necessary for stability in a gravity wall.

The wharf may, however, be intended to be equipped with heavy cranes and warehouses and in such a case it will be necessary to carry the super-imposed loads on bearing piles. Consideration would then require to be given to the erection of a piled wharf along the front of the shore or river bank with a pitched slope facing to the latter as an alternative to the vertical sheet piling along the wharf frontage.

At sites having a large tidal range the adoption of measures to ensure that the water level behind the wall or sheeting rises and falls concurrently with that in the stream will reduce very appreciably the outward pressure to be resisted.

While the foregoing remarks apply to all localities where skilled labour and the necessary materials can be made available without undue expense, in certain localities abroad it may be found that other types of wall construction are in fact the most economical, an example being concrete block construction which can be carried out by largely unskilled labour.

Breakwaters

It should be unnecessary to remark that breakwaters are particularly unsuited for experiments in economy unless very careful consideration has been given to all the relevant factors. The tremendous forces exerted by the waves have produced extraordinary effects in the bodily movement of cyclopean blocks and other components and there have been many unfortunate examples of expensive failure even where the designs were produced by capable engineers with no incentive to economy beyond that activating all conscientious members of the profession. Design should be preceded by a careful study of all available records of waves, winds and currents at the site to arrive at an assessment of the maximum forces which the breakwater will be called upon to resist. Numerous attempts have been made to produce formulæ giving the

pressure exerted by waves of given heights and possibly the most precise of these is that of Monsieur Sainflou. Its application is however, limited to vertically sided breakwaters in sufficient depth of water to preclude the waves breaking, and the more usual problem of assessing the effect of breaking waves still awaits a solution.

Having thus clarified the requirement the type of wall must be chosen most suitable to the circumstances. Where the site and other conditions permit, a mound which by the nature of its construction will disperse the forces of the waves rather than offer to them a solid face may be considered the best answer and a sloping bank of pell mell blocks has been adopted in many instances for this purpose. A recent development in this respect is the invention of a concrete block known as a " Tetrapod " which may be described as a concrete body having four tapering legs projecting in different directions. It is claimed by the inventors that the Tetrapods, the size and weight of which can be made to suit the site conditions, when deposited in place interlock and produce a surface which being of an " open " texture prevents the building up of destructive high pressures in the components of the breakwater. Between the wars when extensive reclamation was in progress in Holland, the Dutch produced a new form of breakwater of which there is now at least one example in the United Kingdom, which was composed of a pumped sand hearting contained between clay banks which, where exposed to wave action, were protected by squared stone pitching. The cross section of the breakwater was arranged so as to allow the waves to expand their energy in running up its sloping face.

There is ample scope for investigation and research into breakwater construction and whereas in the past there has been a tendency to employ boulders and concrete blocks of increasing weight and to introduce measures designed to make the structure more monolithic, economy may well lie in a different method of approach.

In conclusion it cannot be too strongly emphasised that adequate time must be allowed the engineer if the most satisfactory and economical design is to be produced. There are occasions when speed is an over-riding factor, but if it is agreed that the best service that can be rendered to the client is to meet his requirements with full regard for economy in first cost and subsequent maintenance there must be a detailed investigation of site conditions and careful consideration of the form the structure should take and the method of construction, preliminary to entering upon the detailed design. Time thus occupied is time well spent.

COMPARATIVE FORMS OF CONSTRUCTION

By L. R. Creasy

This part of the paper is concerned with investigations which have been made and of examples of work carried out in alternative forms of construction of low steel content, and it is intended to give an indication of the material content and where possible of other relative factors such as cost and ease of construction.

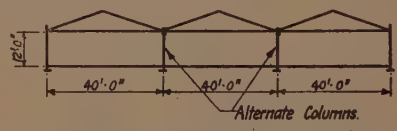
The figures for costs are given for the time at which these were current and must in some cases be brought up to date by the reader, as appropriate.

As has been repeatedly stated elsewhere, the most fruitful opportunities to achieve maximum economies are to be found at the planning stage. In trussed roof framework generally, a basic principle obviously applies in that the greater the area of unimpeded floor space the more expensive the construction will be in both material content and in cost, but superimposed on this are the alternative possibilities of varying the centres of lattice girders and trusses with corresponding variation in the distribution of material between purlins and supporting framework. In a simple truss and purlin roof th familiar economic principle applies of, " not carrying th load twice " in that the closer the truss centres the lowe the content of the purlins and that of the overall stee content. A practical limit is, of course, reached whe the trusses cease to use practical stock sections and thi has been found at truss centres of just less than ten feet

Where valley beams or girders are required to increas the unimpeded covered area, the rate of increase of stee content with increase in span of girders and spacing o columns is clearly shown in Fig. 1 already referred t which indicates also the effect of the form of constructio on the relative steel content.

In terms of cost the conventional bolted angle truss an joist cannot, in general, be improved upon, but reduction of the order of 15 per cent. in weight can b made if the same framework is shop welded and as muc as 40 per cent. in weight by the substitution of welde tubular construction. At the other end of the scale, th undoubted amenities of the longer span space frame roo using open welded vierendeel girders can only be mad with increase in cost.

An illustration of these factors as applied to a typica programme of single-storey storage buildings of thre 40 foot span bays designed in five alternative forms o construction is shown in Fig. 3. In this example th

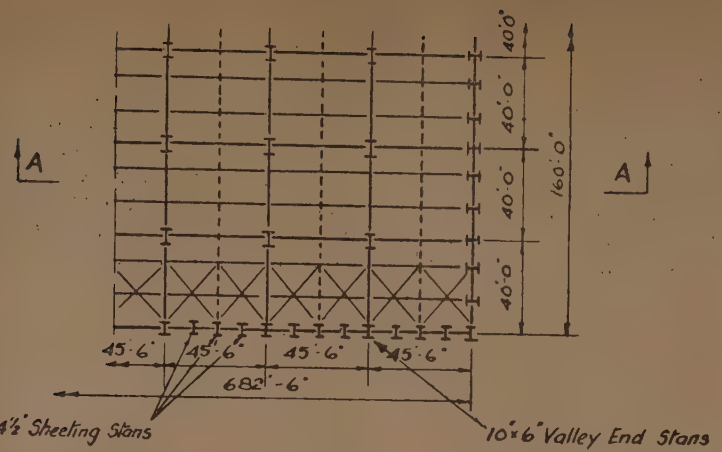


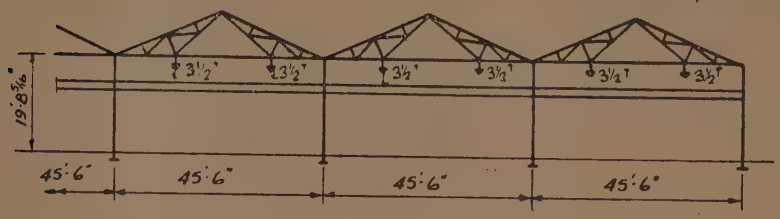
Factors for "steel content" and "unit cost" are given as an average per square foot of floor area covered. Date: 1951.

TYPE.	STEEL CONTENT (lb.)	UNIT COST. (shillings)
TUBULAR CONSTRUCTION THROUGHOUT.	2·6	2·3
STRIP STEEL FRAMES:		
(a) WITH JOIST STANCHIONS	3·0	2·4
(b) STRIP THROUGHOUT.	2·9	2·6
LATTICED PORTAL FRAMES.	4·1	2·5
CONVENTIONAL STEEL CONSTRUCTION.	4·7	2·7
REINFORCED CONCRETE CONSTRUCTION.	1·2	3·3

Fig. 3—Storage Shed Programme. Comparative costs and steel content

lowest steel content is achieved by employing welded steel tube truss and post construction and is some 60 per cent. of that of a similar design using conventiona bolted steel angle work; with cold strip steel con struction, very close to that of the welded tubes in weight of material and cost. From the factors quoted it will be seen that the cost of the tubular work is some 90 per cent. of that of the steel angle work but this applies to the particularly favourable circumstances of a repetitive programme and normal experience suggests that the cost of the tube work generally will be approxi mately equal to that of the conventional forms of con struction, where there is less repetition or the work is





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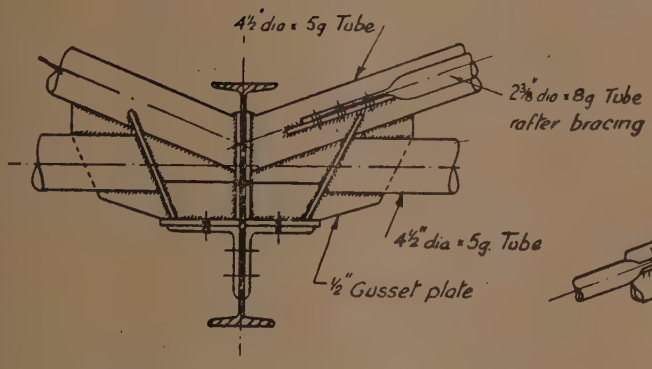
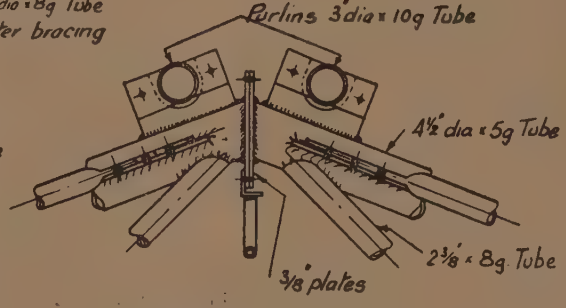
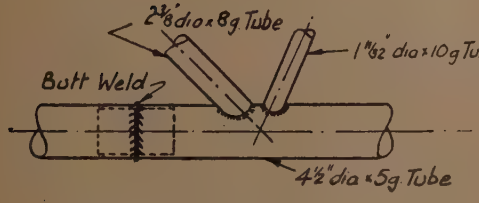
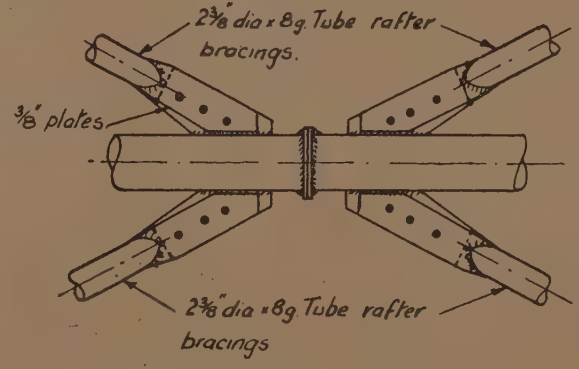


DETAIL OF VALLEY AT BRACING BAY





DETAIL OF JOINT IN TIE



DETAIL OF APEX AT BRACING BAY

TYPE	STEEL CONTENT	UNIT COST
CONVENTIONAL	3 9 lb	2·1 Shillings
TUBULAR	2·4 lb	2 5 Shillings

Factors for 'steel content' and 'unit cost' are given as average per sq ft. of covered floor area for roof construction only Rates include one priming coat Date Mid 1952

Fig. 4—Roof framing

ore complex, or where the conditions are otherwise ore adverse.

An example of such an unfavourable case is shown in g. 4 which refers to a single storey pitched roof industrial building with travelling cranes in which a substitution has been made of welded tubular roof framework to pick up the connections for the conventional joist columns and beams as originally planned. The

trusses are of two different spans and are designed in each case to carry two 3½ ton runways placed symmetrically at the main node points in the bottom tie. The tubular construction shows a saving in metal of approximately 40 per cent. with an overall increase in cost of nearly 20 per cent. as compared with the conventional riveted steel angle latticed framework. It will be seen that the detail, generally, is more complex and therefore, more expensive than is necessary in this type of work due, in some measure, to restrictions imposed by the original design. At the present time the balance in cost between these two forms of construction is probably made less favourable by the common procedure of substituting available sections which appears to have become almost part of our modern constructional practice.

It has been found that the erection of tubular roof framework tends to take somewhat longer than the conventional form, for a number of reasons. A somewhat different technique is required in that, although the tubular trusses are stiffer and lighter in lifting and handling than angle trusses the erectors are not able to walk along the tubular rafters as readily as they would in normal practice and this leads to a somewhat more distant erection technique. Likewise the fixing details tend to be more complex and a site welded joint of the sleeve type in the bottom tie where trusses are delivered to the site in halves, requires the use of a crane for the manipulation of the work. It is probable that costs are reduced to a minimum by a part welded and bolted construction in which the work is fabricated in welded units as large as possible prior to erection in place with bolted connections. A further reduction might be made in suitable circumstances from reduced transport charges achieved by the setting up on site, of a temporary workshop where delivery is taken of the tube direct from mills, for fabrication at site at ground level.

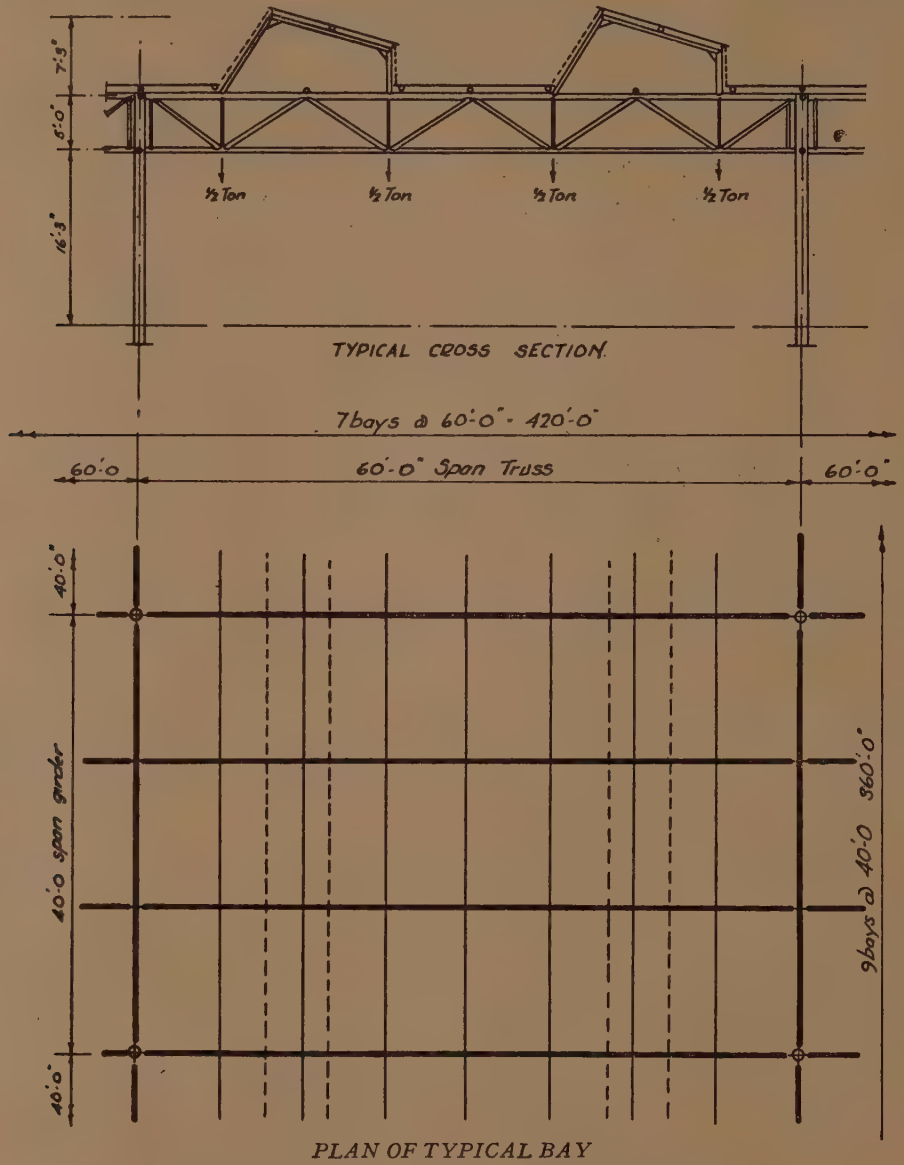
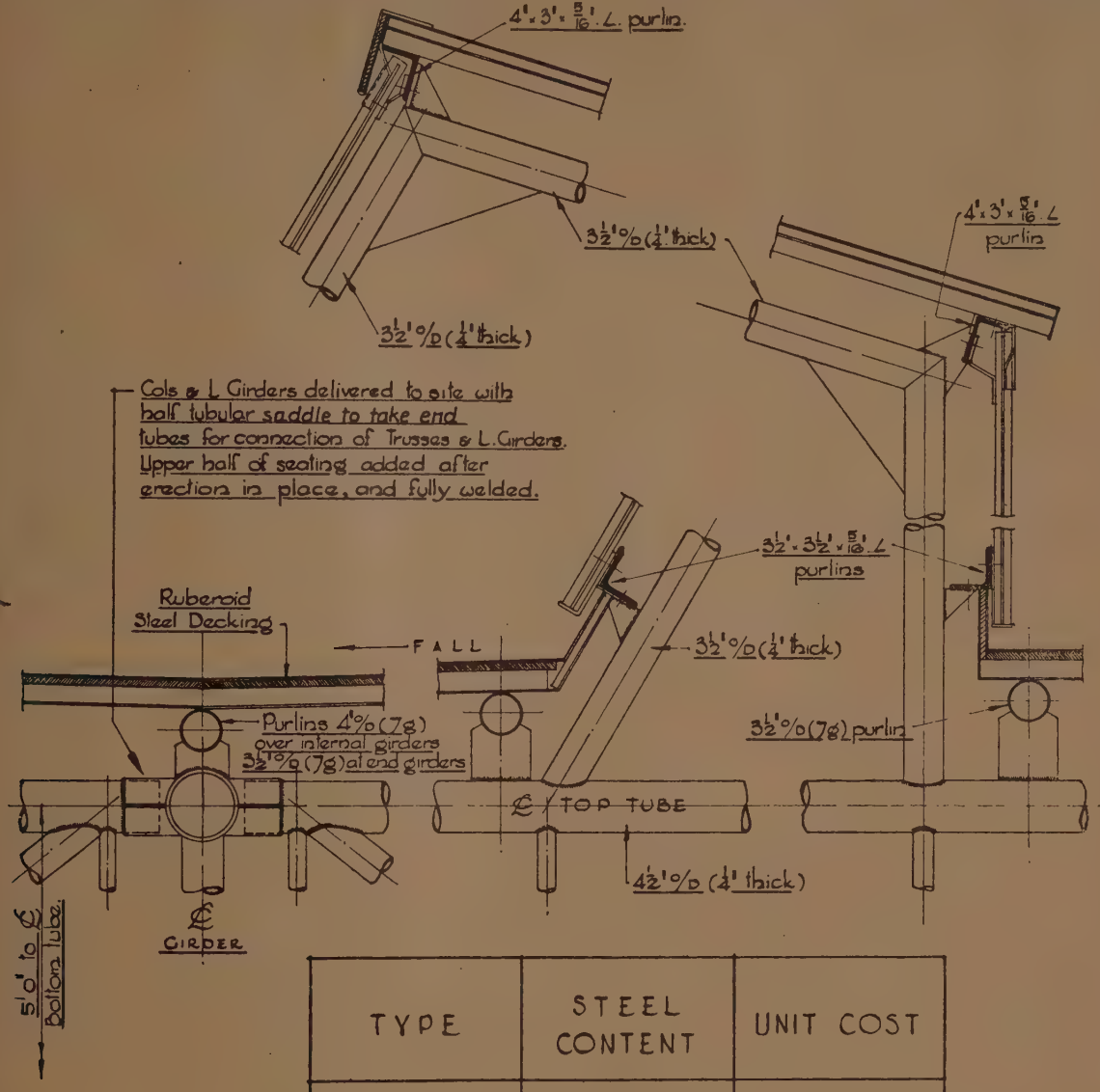


Fig. 5—All welded tubular construction—layout

We are indebted to Mr. E. Bedford for a particularly neat solution of the problems of tubular construction illustrated in Figs. 5 and 6, which shows a single storey factory building of fully continuous all welded lattice tubular work. The arrangement of the monitor roof and layout of the frame is planned to suit the architectural and production requirements, but the symmetry and lightness generally of the tubular sections accompanied by the absence of gusset plates and bolts, achieves a pleasing light-weight appearance, whilst the painting and maintenance of the structure is minimised due to the smaller surface area of tubes and the absence of ledges and inaccessible surfaces inherent in trusses formed of angle sections.

This tubular structure is estimated to be some 30 per cent. less in weight and 10 per cent. less in cost than a conventional design using riveted angle sections; as much of the structure was welded at the works prior to delivery to site and no bolts were used for erection purposes. The erection of the structure is a simple operation facilitated by the welding of semi-circular saddles to the columns which provide self-aligning seatings for the trusses and girders and form, with the addition of a similar top saddle the permanent connection. The whole structure of some 460 tons of steel was erected, completely welded in five months.

Before leaving the question of storage buildings it might be of interest to compare the alternatives illustrated in Fig. 1 with that of the famous Cambridge design as based on the plastic theory and as shown in Fig. 7. The steel content for this fully welded three span shed of 60 foot span by 24 foot centres of columns is only 3 lb. per square foot of covered area and is fully described in THE ROYAL ENGINEERS JOURNAL of September, 1951.


4" x 3" x 5/16" ∠ purlin.
3½" ø (¼" thick)
3½" ø (¼" thick)
4" x 3" x 5/16" ∠ purlin
Cols & L Girders delivered to site with half tubular saddle to take end tubes for connection of Trusses & L. Girders. Upper half of seating added after erection in place, and fully welded.
3½" x 3½" x 5/16" ∠ purlins
Ruberoid Steel Decking
FALL
3½" ø (¼" thick)
Purlins 4" ø (7g) over internal girders
3½" ø (7g) at end girders
3½" ø (7g) purlin
℄ TOP TUBE
4½" ø (¼" thick)
℄ GIRDER
5'0" to ℄ Bottom Tube.

TYPE	STEEL CONTENT	UNIT COST
CONVENTIONAL	9·0 lb	5·8 s.
TUBULAR	6·3 lb	5·2 s.

Factors of 'Steel Content' and 'Unit Cost' are given as an average per square foot of covered floor area.

Fig. 6—All welded tubular construction—details

The last example of framed roof construction as shown in Fig. 8 compares a typical north light factory roof in conventional bolted angle construction with one using castellated beams and is inserted as a reminder that the search for alternatives often produces surprising results. The scheme using castellated beams shows a somewhat greater steel content than the conventional—at lower cost!

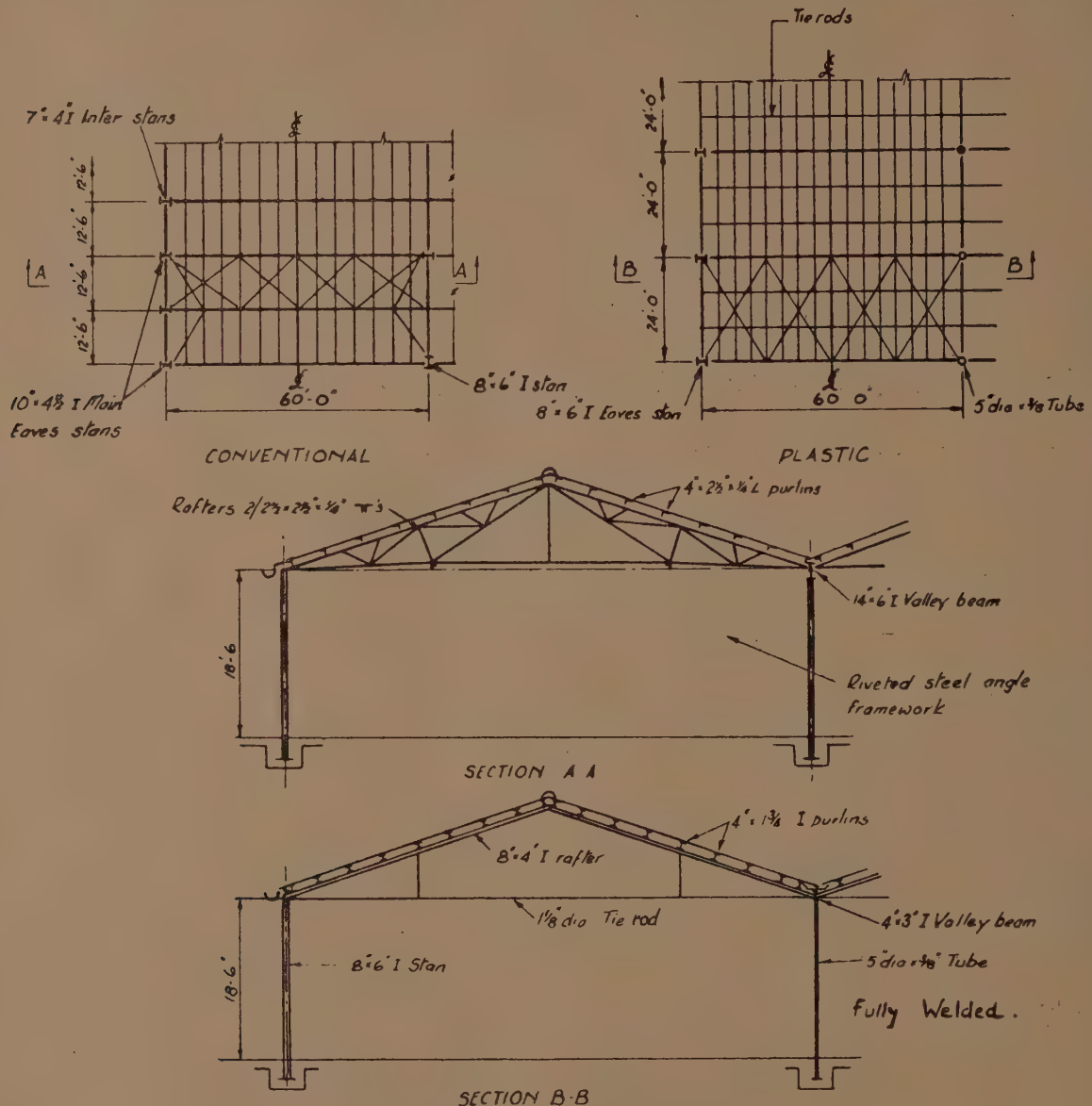
An example of the economical use of light latticed steel framework required in the many miscellaneous structures needed for modern large scale industrial developments is shown in the pipe bridges illustrated by Figs. 9 and 10. The former is in welded tubular work and the latter is an interesting example of the modern use of one of the oldest structural forms—the trussed beam.

Turning now to the question of alternative forms of multi-storey framed construction, the most obvious field of enquiry lies in the conventional cased steel joist frame in which by the usual methods of design, the contribution of the casing to the overall strength is not fully taken into account.

An analysis of a heavy industrial framed building is shown in Fig. 11, in which a comparison is made between a cased steel and a reinforced concrete framed structure and each in turn compared with a modified part load bearing form of construction. The depth of floor construction for the reinforced concrete frame has been assumed as being of the order of one third greater than that required for the steel frame and this, of course, results in a greater overall height of building for the same floor clearance.

As would be expected; the building costs, excluding the actual cost of steel are greater for the reinforced concrete building by virtue of the increased overall height but this increase is more than offset by the saving in cost of steel reinforcement so that the overall cost of the reinforced concrete buildings is a little less than those modelled on a conventional steel frame.

In examining a typical multi-storey office block, an example of which is shown in Fig. 12, it will, of course, be found that the reinforced concrete frame often suggests a rearrangement in the planning. In this case the lighter



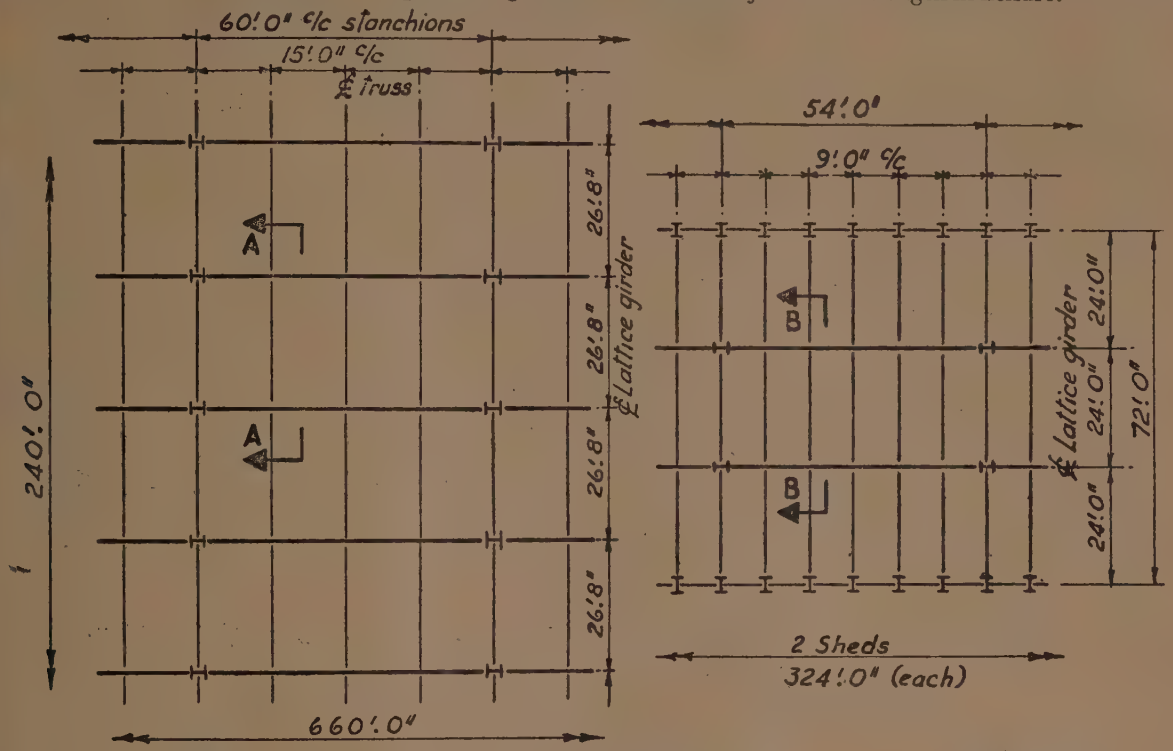
TYPE	STEEL CONTENT
CONVENTIONAL	4 6 lb
PLASTIC	3 0 lb

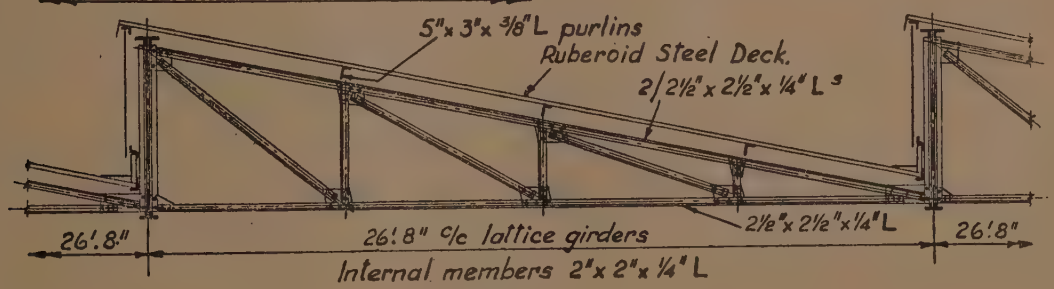
Factors for 'steel content' are given in lbs per sq ft of floor area covered.

Fig. 7—60 ft. span shed

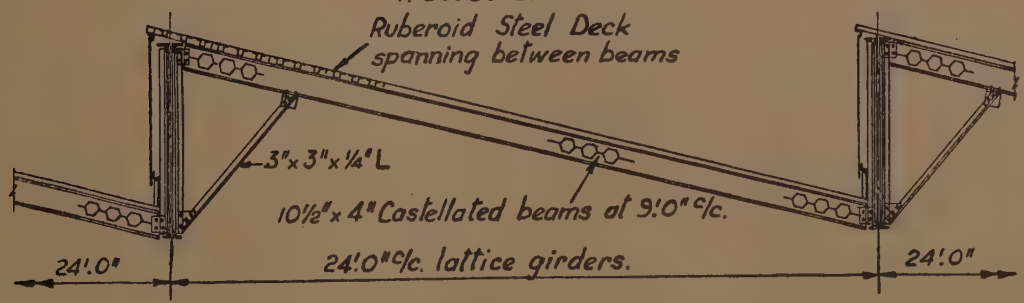
.ds to be carried do not require an increase in the height the building and from the architectural point of view e increase in size of reinforced concrete columns is ssibly counterbalanced by the advantages of trans- rse floor spanning accompanied by flush ceilings and eater freedom of internal planning.

In the realms of foundation work generally; the miliar basic structural problem of preserving the balance between load and resistance emerges with new complexities; each containing a varying measure of uncertainty in their magnitude and consistency. The greatest contribution which can be made to economy will no doubt follow from a correct estimate of these factors and in a proper choice during the development of the design between a structure whose stability generally, is invoked by mass or strength in flexure.





SECTION A-A



SECTION B-B

TYPE	STEEL CONTENT	UNIT COST
Conventional	4·8 lb	2·9 s.
Castellated beams	5·0 lb	2·7 s

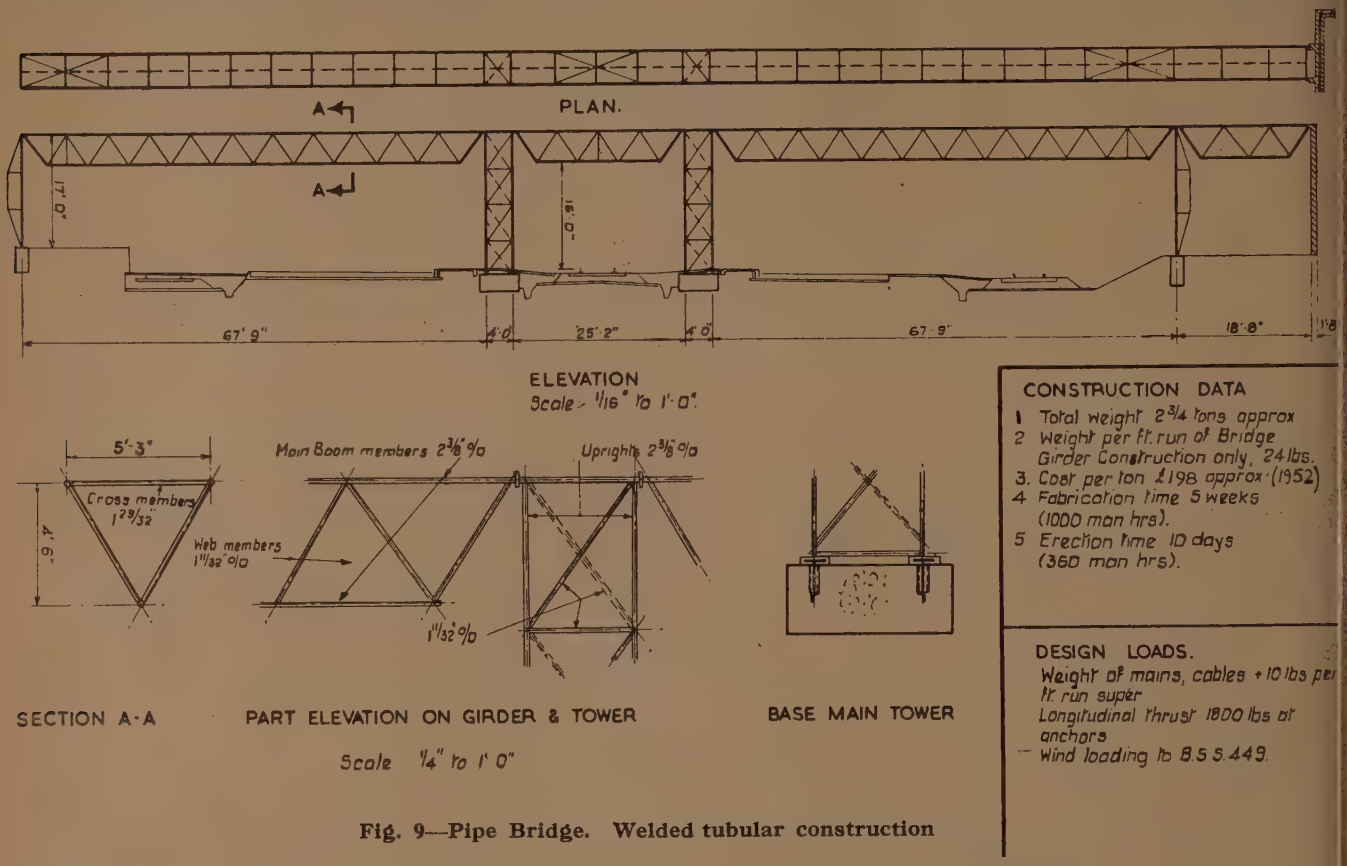
NOTES :-

Factors of 'Steel content' and 'Unit cost' are given for an internal bay per square foot of covered floor area.

Date – Mid. 1951

Fig. 8—North light roof

A fundamental problem in basement construction, for example, which is typical of these uncertainties, is to be met at the outset in deciding on the magnitude of the water head in a clay sub-soil for which the retaining walls are to be designed and on how far the pressure from the same head can be assumed to remain constant over the basement slab. A not uncommon assumption is t assume the basement to float in a film of water under pressure head taken from the surface of the clay and hov far this severe condition is justified depends on th relative factors of sub-soil and terrain. It might b worth noting in passing, that in a number of such cases i



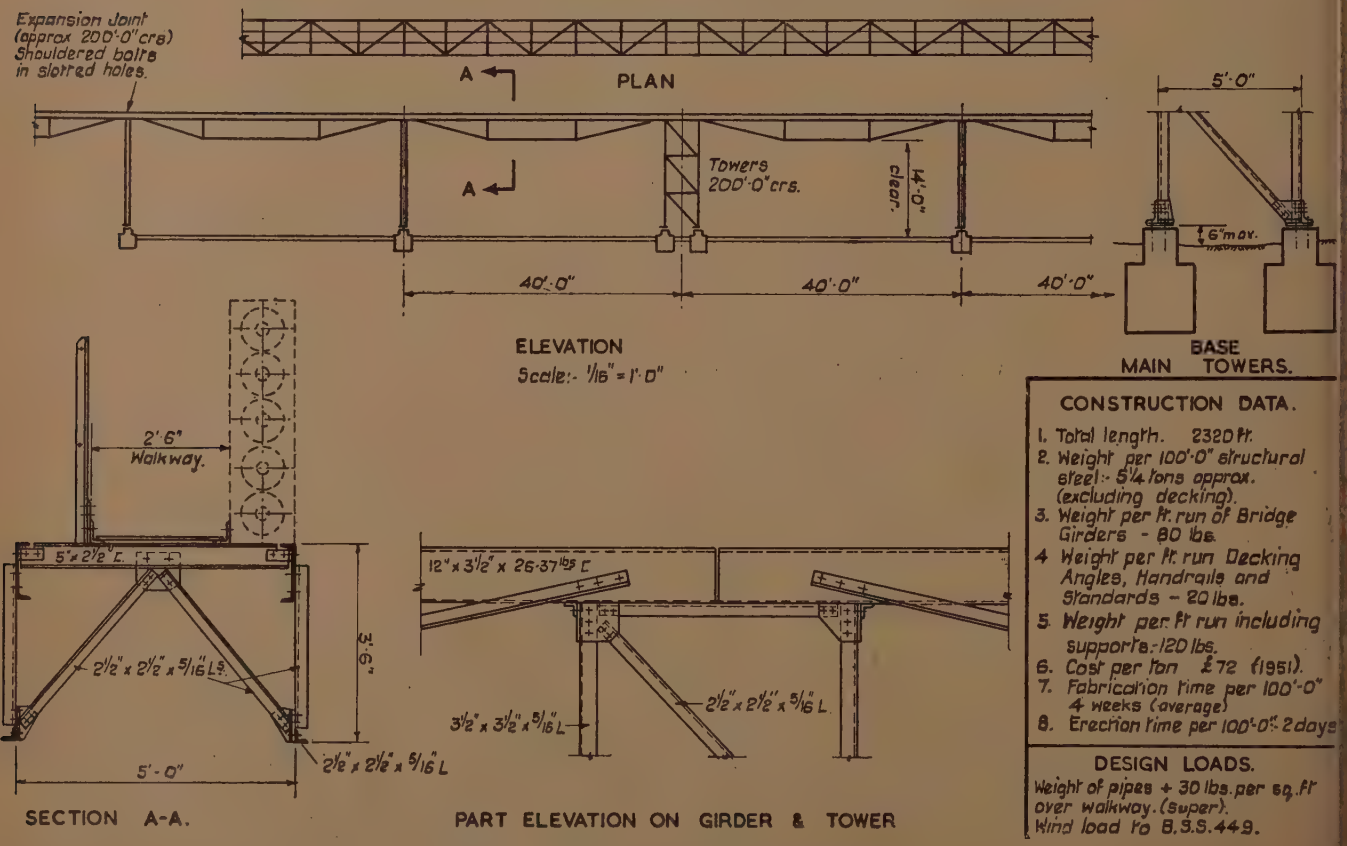
CONSTRUCTION DATA

1. Total weight 2 3/4 tons approx
2. Weight per ft. run of Bridge Girder Construction only, 24 lbs.
3. Cost per ton £198 approx. (1952)
4. Fabrication time 5 weeks (1000 man hrs).
5. Erection time 10 days (360 man hrs).

DESIGN LOADS.

Weight of mains, cables + 10 lbs per ft. run super
Longitudinal thrust 1800 lbs at anchors
Wind loading to B.S.S. 449.

Fig. 9—Pipe Bridge. Welded tubular construction



CONSTRUCTION DATA.

1. Total length. 2320 ft.
2. Weight per 100'-0" structural steel :- 5 1/4 tons approx. (excluding decking).
3. Weight per ft. run of Bridge Girders - 80 lbs.
4. Weight per ft run Decking Angles, Handrails and Standards - 20 lbs.
5. Weight per ft run including supports :- 120 lbs.
6. Cost per ton £72 (1951).
7. Fabrication time per 100'-0" 4 weeks (average)
8. Erection time per 100'-0" 2 days

DESIGN LOADS.

Weight of pipes + 30 lbs. per sq. ft over walkway. (Super).
Wind load to B.S.S. 449.

Fig. 10—Pipe Bridge. Riveted construction

the author's experience, where pitot tubes have been cast into the base slab as tell-tales, no such severe condition has ever been recorded.

The problem of design in mass or flexure will obviously involve economies in one material at the expense of another and resolve itself into one of comparative assessment. An example of such alternative treatment of a typical basement designed against a specific water head is shown in Fig. 13. The lower slabs are designed in reinforced concrete to span between the column anchorages and alternatively in mass concrete as dead weight. The vertical walls are designed as propped cantilevers in reinforced concrete and alternatively in mass brickwork on the basis of a flat arch as indicated.

RELATIVE BUILDING COSTS

COSTS ON SAME BASIC UNIT OF £100,000 ORIGINAL COST.	STEEL FRAME WITH BASEMENT.	R.C. FRAME. WITH BASEMENT.	STEEL FRAME. NO BASEMENT.	R.C. FRAME. NO BASEMENT.	EXT. BRICK. INT. R C. FRAME. NO BASEMENT.
BUILDING [LESS STEEL]	75,000	80,000	64,000	67,000	69,000
STEEL IN FRAME.	25,000	13,000	16,000	7,000	5,000
TOTAL	100,000	93,000	80,000	74,000	74,000

Prices as at 1-1-52.
Framed Bays at 12'-0" centres
Clear inside dimensions: 48'-0" wide x 12'-1½" floor to floor headroom
R.C. Frames as compared with Steel Frames-
1'-0" increase in gross floor to floor dimension
Floors designed to 180 lb. p.s.f. super

Fig. 11—Heavy framed building

This latter approach to the design of a brick wall introduces more economical proportions than those determined on the normal beam hypothesis and accords better with the results of tests and experience of their stability.

These examples are chosen at random and over as wide a range as possible in the limited time available, as illustrations of alternatives to the conventional approach. This list does not pretend to be exhaustive and is presented less as a catalogue than as an invitation to the reader to extend the record from his own experience.

DISCUSSION

THE PRESIDENT, proposing a vote of thanks to the three authors of the Paper, said the Institution was grateful to them for having written and presented it and for the illustrations they had shown.

There was no need, he said, to emphasise the importance of economising in materials at the present time. Mr. Gardner's suggestion to make a comprehensive series of tests on members and complete structures was a useful one and if we did that we should obtain some most valuable results which would add much to our knowledge.

Expressing agreement with all that Mr. Morton had said, the President commented on the suggestion that there should be research into the construction of breakwaters. He recalled that that suggestion had been made in a paper before the British Association in 1930. He did not know whether this meant that we had made little or no advance in breakwater construction since 1930.

Mr. Creasy also had given much useful information in a most interesting manner.

The President again expressed gratitude to all the authors on behalf of the Institution.

Mr. T. BEDFORD (Member), congratulating the authors on a most interesting paper, said it contained some enlightening and very valuable information on the economics of building structures, and it gave what was so often lacking from some of our technical papers, namely, the relative costs of the various types of structures, without which no true assessment could really be made of a design.

The shortage in the supply of any particular material, he continued, might well be just a passing phase, but economy in the use of materials and labour was something which should always be uppermost in the minds of engineers; in fact it was a major part of the art of the engineer to provide a structure which not only suited its purpose but was also economical.

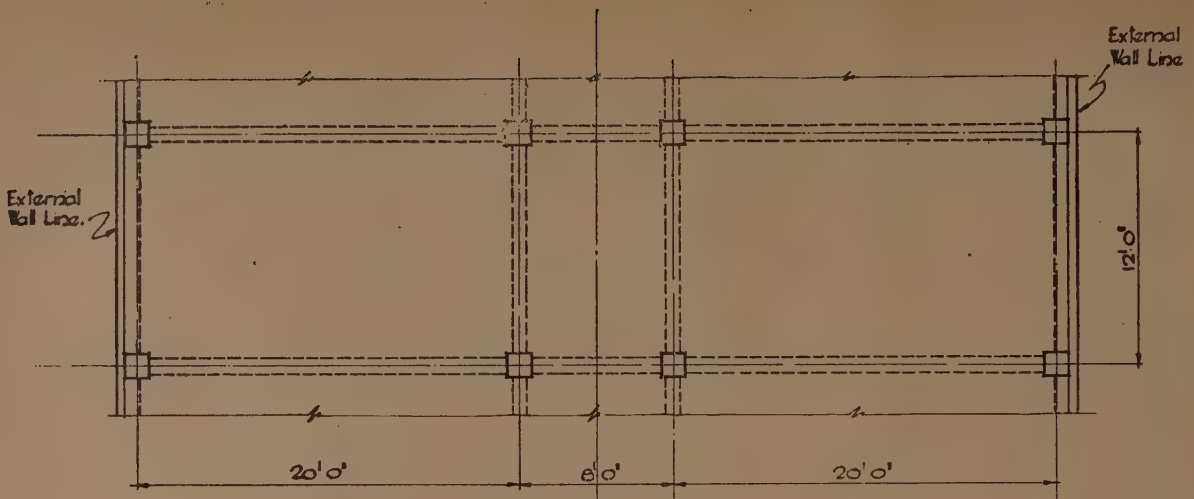
The paper seemed to give rather a jolt to the economic stability of the conventional steel framed bolted or riveted structure, for it would be seen from the examples shown that in the single-storey factory structure the use of tubes showed up to 40 per cent. saving in the amount of steel used and at the same time a saving in cost of up to 15 per cent. There were also examples of multi-storey reinforced concrete buildings, and there again the savings effected could be something like 50 per cent. in steel and 10 per cent. in cost. He was quite sure there was a place for all types of construction. The main consideration, the great aim of the engineer, was to choose the right construction for the particular job in hand.

His particular interest on that occasion was in the examples of tubular construction shown, and he illustrated some further examples of such construction to supplement those given in the paper.

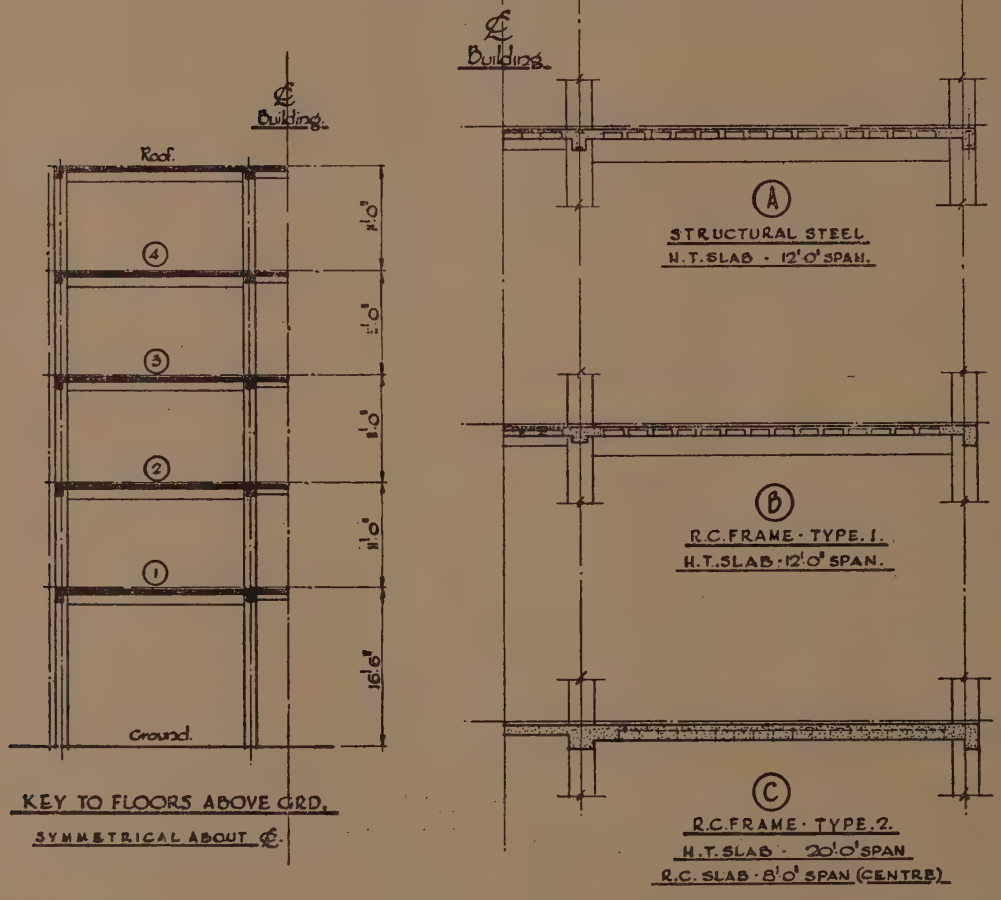
The first was a drawing of the monitor type single-storey factory building referred to by Mr. Creasy in the paper. It was about 420 ft. long and 360 ft. wide. The drawing showed the 60 ft. span parallel boom trusses with two portal frame type monitors. The valley girders were of 40 ft. span; so that there were columns at 60 ft. by 40 ft. centres. The trusses were designed to carry an additional 2 tons each distributed on the bottom chord and they each weighed 16 cwts. The weight of the structure over the internal bay was approximately $5\frac{3}{4}$ lb. per sq. ft. That might not appear to be a particularly economical weight, but it was economical for that type of structure, and north light roof lighting through the monitors was demanded in that case.

Mr. Bedford then showed the structure in course of erection. The 60 ft. trusses were lifted into position very easily by a mobile crane. All the columns were erected first, of course. The picture showed the cleats on the columns, which cleats were half tubes, semi-circular, so that erection was very simple; the crane simply lifted the trusses into the cleats and they were self-aligned, without further need of support.

The structure was completely site welded. There were no bolts whatever in it, not even for erection purposes, except the anchor bolts to hold the columns in position. The general anatomy of the structure looked very clean and tidy.



TYPICAL BAY



KEY TO FLOORS ABOVE GRD.
SYMMETRICAL ABOUT ℄.

TYPE	STEEL CONTENT
A	12·3 lb.
B	4·8 lb.
C	5·75 lb.

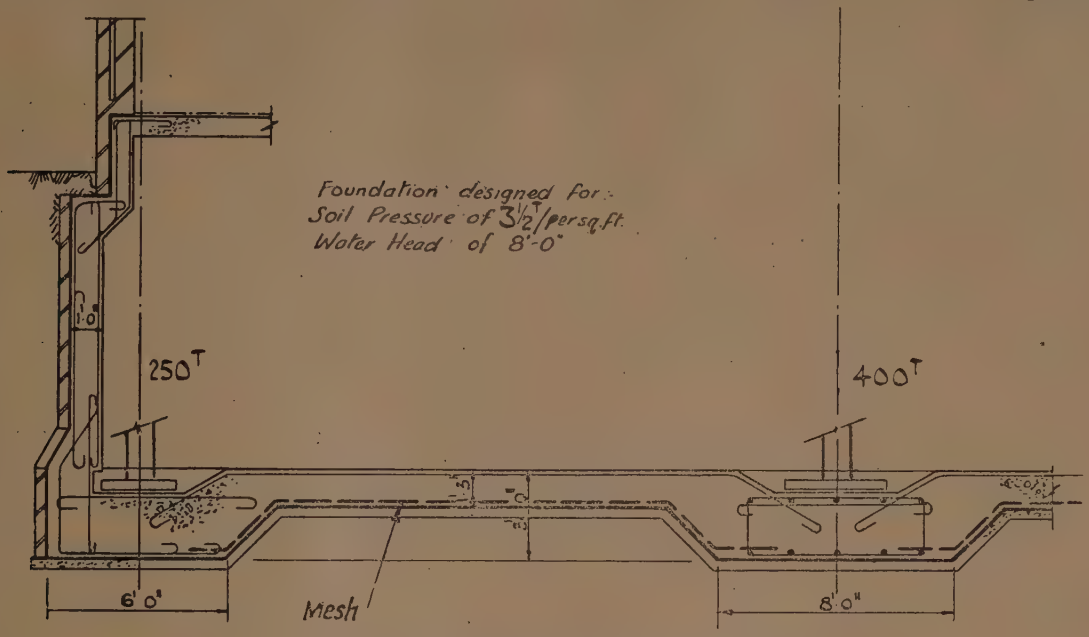
DESIGN CONDITIONS.
Live Load 50 lb/sq. ft.
Screeds & All finishes, Etc, 47 lb/sq. ft.
ALLOWANCE FOR PARTITIONS.
30 lb/sq. ft of Partition between outer and inner columns.
50 lb/sq. ft of Partition between inner columns along corridor.

Factors for 'Steel Content' are given as an average for one intermediate story including columns, per sq. ft. of covered floor.
'Steel Content' of type A includes Structural Steelwork, & Rod Reinforcement in floors and casings.

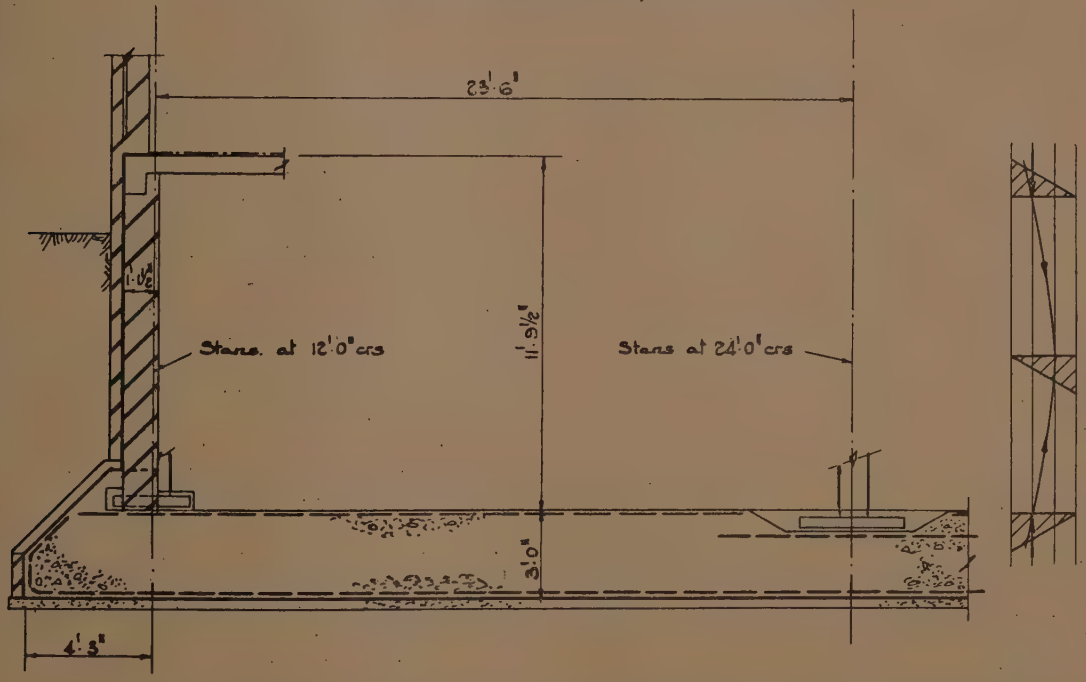
Fig. 12—Framed office building

The next illustration gave a closer view of the 60 ft. trusses with the monitors and the 40 ft. valley girders, showing very clean lines. There were no gusset plates, but all tube to tube connections, except for the small stiffening plates at the top of the monitors. It was a very slender looking structure. The trusses and the 40 ft. girders were fabricated at the workshops, and the connections to the columns were welded on site. The purlins were tubes, except for the glazing purlins, and the tubes were welded in continuous runs.

The next slide showed the connection of the columns with the semi-circular cleats, and a top half circle of



R.C. RETAINING WALL, BASE & SLAB



MASS CONCRETE BASES & SLAB WITH BRICK RETAINING WALL

TYPE	STEEL CONTENT	CONCRETE	BRICK
R.C.	8·5 lb	2·5 ft.	—
MASS	1·0 lb	3·0 ft	0·5 ft.

Factors for 'steel content' are given as average per sq. ft. of covered area.
For 'concrete' & 'brickwork' as an average thickness per sq. ft. of covered area.

Fig. 13—Basement construction

cleats added afterwards to complete the job. The only temporary support for holding up the structure were a few bonds; no guys or ropes were required to keep the parts in position while welding.

Then followed an internal view of the factory, showing the nice even lighting. It looked very light and clean, and when inside it one did not have the feeling that was often experienced in single-storey buildings, that there was a lot of steelwork over one's head.

This was followed by a drawing of another factory of tubular construction. A cross-section on the left of the slide showed a series of 40 ft. butterfly type roof trusses supported on trussed girders of 120 ft. span. At the sides were two 60 ft. spans. The girders, in addition to the roof load they carried, would carry an equivalent load of about 8 tons distributed, and the butterfly type roof trusses would each carry a distributed load of about 4 tons on the bottom chord; so that the suspended load was quite appreciable as compared with the normal live load. The weight of the structure over the internal bay, 120 ft. by 40 ft. was 6¼ lb. per sq. ft., so that it was considerably lighter than a conventional form of riveted and bolted structure. It had tube to tube connections without gusset plates. An interesting point about the factory, which was in course of erection, was that practically all the fabrication and welding was done on the site; the tubes for the girders were sent to the site cut to length, and they would be welded together there.

The next illustration showed the 60 ft. bay truss where it joined the latticed girder, and then the butterfiy truss continued over the internal bay. The roof was of very light and elegant form and a rather pleasing anatomy.

Mr. Bedford then showed a similar sort of truss, except that the truss connections to the columns were made by means of bolts. It was sitting on the top of a tubular column, and the bottom chord was joined by means of a flange. It was very simple to erect and again had a pleasing appearance.

Mr. A. P. MASON (Member), submitted written comments on the paper, in which he referred to Mr. Gardner's introductory remarks summarising structural economy as " the amount of money which is exchanged in the process of getting what we want " and relating it to the total amount of human energy used. Every engineer who had practised for more than 25 years was very much aware of the increase in the proportion of human energy required to produce the design and working drawings before the builder could make a start. When imposed loads were high and working stresses low, most structures could be designed by quite simple rules. In our modern structures we provided probably only one third of the strength formerly customary in our office buildings and flats and as a consequence there were not so many short cuts in the design, which must be carried out with considerable mathematical effort. As a result a substantial portion of the material saving was expended in the human effort of design and since the structural engineer was still remunerated in terms of a percentage of the total cost he must nowadays work much harder to achieve a given result. This might in part explain the serious shortage of designing engineers and draughtsmen which often exerted a delaying action in the construction of a building. When we considered the revision of the working stresses in the Codes of Practice, we should bear this economic aspect in mind and also have due regard for Mr. Gardner's cautionary words " every step must be taken with extreme caution."

Mr. Mason agreed that in a structural sense there was no case against employing load bearing walls, rather the reverse, because we must have a wall to keep out the weather and why not make it serve two purposes, bu wartime experiences showed that load bearing wa structures had little resistance to blast and it has bee proved that only framed structures could withstand th atom bomb. The state of the world did not yet justif any optimism in the matter and it was futile to mak elaborate plans for Civil Defence if we were putting u death-trap buildings.

Mr. Moreton observed that when reinforced concret jetties began to replace timber the arrangement members was a replica and there was the disadvantage carrying out the work between tides. Mr. Mason ha always adopted precast piles, horizontal braces an diagonals so that only the junctions required to be cast *situ*. This system was easily adjusted to incorporat pre-cast concrete units. It was however, hardly fa to cite the unfortunate results of the highly competitiv state of the reinforced concrete industry in its earl stages as an argument for prestressed concrete, no or would think of allowing only ¾ in. cover to the rei forcement in present day work.

The comparative costs of various methods of storag shed construction given by Mr. Creasy in Fig. 3 would b more useful if some information was provided on the un rates employed in arriving at the costs. The figur themselves were not free from error. The total weight steel for tubular construction should be 31 tons and th cost £2,980 based on the given rate per ft. super. In th case of conventional steel the total weight should b 53 tons. The weight of steel for a reinforced concre shed was extraordinarily low unless it was assumed tha only the framework of a building with a light coverin was included. The comparisons would be more com plete if figures were given for the consequent maintenan costs in which case reinforced concrete would show u more favourably.

He considered that shell concrete roofing should b considered in this comparison.

In Fig. 2 the weight of steel required for a reinforce concrete heavy framed building had been reduced at th cost of making the floor depths one third greater tha cased structural steel. Generally, reinforced concre could be the same depth or very little more than case steel without introducing design difficulties. The com parison must be regarded only from the point of view using the least quantity of steel because the cost of rei forced concrete must have been increased by th additional concrete, formwork and walling. The clai made that this additional cost was more than offset b the saving in the cost of steel reinforcement was not accordance with normal experience. As the particul example shown was only two bays wide it did not perm comparison with a reinforced concrete flat slab schem The imposed load of 180 lb. per sq. ft. indicated a buildin of the warehouse class in which case columns at 12 f centres would not be regarded as very convenient.

Purely from the point of view of detail, Fig. 13 shou not be taken by students as a model for their own wor If the rods shown bent round the inside of obtuse angl were under tension then they would burst away th concrete. If they were not in tension then they were n required at those particular points.

The paper would be more complete if more of the usef recommendations given in Steel Economy Bulletin No. were incorporated, such as the use of high tensile ste bars and drawn wire reinforcement. The cost figures f reinforced concrete structures were presumably based ordinary mild steel bar reinforcement, but in view of t Bulletin figures based on high tensile steel might ha been given with advantage.

Economy in design must be considered in relation to the quality of the work on site. The Bulletin stated that "in the hands of experienced designers a stress of 20,000 lb. per sq. in. is reasonable for mild steel bars and up to 30,000 lb. per sq. in. for cold worked bars." In the case of important work where a high standard of supervision was provided no risk was involved other than what must be accepted by lowering the factors of safety. In the case of the smaller building, jobs which made up the bulk of the volume of reinforced concrete construction the same supervision was not provided and there were times when one felt that even the stresses allowable in the Code of Practice should not be adopted in full.

Mr. E. H. BLOOMFIELD (Associate-Member), after commenting on the great interest of the subject and the valuable information that had been given, endorsed very heartily some of the points raised in the paper.

First he commented on the general point concerning the length of time allowed for the preparation of the design and details. He felt that one of the most important functions of the Institution of Structural Engineers should be to endeavour to get clients or architects into a frame of mind whereby they would give the unfortunate engineer time to prepare details before the job started instead of having to do that towards the end of the job, as so frequently happened.

It was very interesting to observe the introduction of welded tubular steelwork into modern structural methods, and he felt there was a tremendous future in that type of construction. It had many advantages, a major one being its flexibility in construction; by that he meant that it could be welded and constructed with comparatively small plant, without the tremendous set up of the fabricating shop. Apparently Mr. Bedford made the various parts of his structures on the site as the erection proceeded, which was a very excellent experiment and the length of time required for fabrication was very much less than for conventional steelwork.

One was very interested to hear Mr. Bedford speak of the beam/column connection. Mr. Bloomfield said he had seen a building of rather the same nature in which the connection of the beam to the column was arranged to come on saddles, so that any slight discrepancy in the length of the beam could be taken up. It was the scaffold type of connection, so that there was no difficulty in matching holes and the erection was carried out very easily.

At this stage the President left the meeting to attend another function at which he was to represent the Institution. He said he had apologised to the authors for his early departure, and Mr. Andrews had kindly consented to deputise for him.

Mr. WALTER C. ANDREWS, O.B.E., M.I.C.E. (Immediate Past President), occupied the Chair during the remainder of the meeting.

Mr. A. B. DAILEY (Member), said that a client expected the Consulting Engineer to advise him as to the quickest and the cheapest way to do a job.

He made a plea, however, for keeping in mind the importance of the aesthetic side of engineering, which perhaps was not economy, the subject of the paper, but was indeed very frequently the contrary. Before the first world war he was an architect, and as the result of his training he always kept appearance in mind.

In the case of factories he understood that the managements found it paid them to paint the interiors in cheerful colours; this kept the employees happy, and the managements were repaid by way of better work. The same should apply to the structural work. During the war the Government had required the use of square gussets in roofs, which was a mistake. He always admired Victoria Station roof, where the gussets were very nicely curved; it appeared to him that the designers of that roof had the aesthetics of the construction definitely in mind.

Mr. JOHN KING (Associate-Member), added his congratulations to the authors and made two suggestions.

First, he said the scope of such research as had been done was obviously very considerable. But it seemed that those who were engaged in the smaller offices had to look to larger establishments, such as Government Departments, for the bulk of the research, and as a method of publicising Mr. Creasy's case book, which was an excellent idea, he suggested that the Building Research Station might perhaps do something along the lines of their very excellent Building Research Digest.

Secondly, in connection with effecting economies, we all knew the very bad standard of reinforced work which was done by some of the smaller builders, which rather complicated matters. But he suggested it might be possible that firms which were known to have a very high standard of site supervision could be permitted by the Authorities to use higher stresses, the less efficient being permitted to use only the lower stresses. An economy of 10-15 per cent. might be effected thereby.

Mr. R. G. TAYLOR (Associate-Member), recalled Mr. Gardner's reference to overall economy in energy, and in energy as applied to actual building materials, and said the tubular structure not only effected economics in materials, but also showed economy in the amount of painting required, and so on. His firm had found that, with a 40 per cent. saving in the amount of steel used, they very often achieved 40 per cent. saving in energy, which in the case of a large roof was important.

In regard to aesthetics, generally speaking a tubular structure had not gussets, so that it had a lighter and more pleasant appearance; the general size of the members was smaller and more light was allowed to come into the building.

Referring again to paint and painting, he said that not only was there a saving in the amount of paint used and in the labour involved, in tubular construction, but the structure itself was easy to paint. On the other hand, if we had to paint angles with $\frac{3}{8}$ in. gap between them we should first clean them; but he questioned that we could in fact clean them adequately. If, however, we employed double tubes we took great care to ensure that there was plenty of space between them to allow cleaning; the tubes were also self-draining in respect of both moisture and dirt, and there was not a bed on which corrosive substances from the workshop would rest, substances such as concentrated acids, which would eat into the steel if proper maintenance precautions were not taken.

Dr. E. H. BATEMAN (Member), commenting on a remark by Mr. Gardner that we must not be led too far off the practical track by an elaborate analytical approach, said he had always been rather interested in the analytical approach. For many years we had been inventing new methods of analysis, and he asked if Mr. Gardner would not consider it reasonable that some of us might be put to work to use the analyses, instead of hoping that the junior draughtsman would apply them. We needed a little more standardisation. We could not standardise everything in building construction, of course, but a number of things might be standardised. For example, some attention might be given to the

anatomy and design of standard types of roof trusses of convenient span lengths. Much time and effort might be saved if we had some standard optimum patterns for simple things which could be taken off the shelf. After all, we did not work out everything from first principles, but used the reference books, and we should extend the scope of reference in design.

Mr. GEOFFREY PALMER (Member), mentioned an experience he had had about 15 years ago, which might serve as a comment on Mr. Gardner's point about intersections of members in structures. A new plant house had to be put on the roof of a London theatre. It was found that a lattice girder, having a span of 60 ft. or so, was adequate to carry the extra load, so far as the sizes of the members were concerned, but the intersections at the end of the girder were roughly 10 in. apart on a vertical line through the support. The effect was evident when a line was strung along the end members of the top and bottom booms; the curvature due to local bending could be seen. He put an extra member in the girder, starting from the first intersection point of the bottom boom and connecting to the top boom a foot or two from the end, tightening the new member by warming it up before connecting it. The original bottom boom plus the additional member constituted in effect a bottom boom in the right place. The extra load was then applied quite satisfactorily.

THE CHAIRMAN, commenting on Mr. Gardner's reference to wall panels and the use that could be made of them, said that most engineers were afraid of using them, even if permitted by regulations to do so, because of the danger that particular panels might be moved. There were many cases, however, where the possibility of brick panels being moved was very unlikely. He had had the opportunity to use them structurally in a block of flats in Scotland which had a reinforced concrete frame. In the ordinary way the party walls would have applied a considerable load on the beams at each floor level. He had avoided this by carrying the load of the complete wall right down to the foundation and had taken the precaution of having the wall built as the floors and frame went up, so that the weight of each brick panel did in fact rest on the one below. That had effected a considerable saving in construction; and no harmful effects, as far as he was aware, had resulted.

Most of the Bulletin was concerned with saving in materials; but two important instances had been mentioned, by Mr. Morton in the case of jetties and Mr. Creasy in the case of buildings, of the use of different raw materials, giving a better result and at the same time saving money. He mentioned that because we must be careful what we meant by "economy." Economy in the use of a particular material was a passing phase. At one moment we were concerned about one material, and a few months later we found that we had to worry about some other material. We were in a period of rapidly changing values, particularly as we were not working to a free economy and when artificial values were placed on various materials, and he felt that money economy was the only safe yardstick.

Mr. Gardner, replying to the discussion, said the two cases discussed by Mr. Bedford were very fine examples of economy. It was interesting, however, that when Mr. Bedford had shown him the first one he had taken Mr. Bedford to his office to check up, and found the unit weight 0.80 lb. per sq. ft. in excess of the minimum. The reason was that the trusses had to carry an incidental load of two tons; and that accounted for the difference. The figure of 5.75 lb. per sq. ft. overall weight might ha been 4.95, but for the extra 2 ton loads. In the seco case the figure of 6.25 lb. per sq. ft. would have be 5.60 if the additional loading had not been imposed.

With regard to Mr. Mason's reference to load bearir walls and blast, one could only say that if it were ir perative to deal with blast we could not in general ha load-bearing walls. Incidentally, however, substanti old-fashioned buildings with cellular construction form by division walls, were found to be very resistant to bla in the last war.

With regard to shell construction versus trusses, h Department had found very little, if any, saving there.

The point about reinforced concrete stresses was a so one. Reinforced concrete might not always be very w designed, nor well constructed; but he was strongly of t opinion that nevertheless, it would be difficult to find reinforced concrete building erected so badly that would not be completely safe in normal use at increas stresses.

Commenting on Mr. Andrews' reference to w panels, he said that a particularly good case was t diaphragm panel in a long shed, where, if there were t and bottom members, the wall panels would look aft the rest, and the bricklayer was saved much unnecessa labour in not having to hack the bricks about to fit t bracing.

Coming to Mr. Bloomfield's reference to tubul construction, showing advantages from the point of vie of economy and aesthetics, he said Mr. McMinn (who w present) had gone into the matter very carefully and h produced some very fine details. There was really marked saving, from many points of view. Mr. Gardn suggested that there would be an even greater saving steel if rod tension members were used.

Expressing sympathy with Mr. Dailey's view th engineers should pay more attention to aesthetics, he sa he believed that many structural designers were oblivio of that aspect and knew nothing about the subjec which was bad. Tubular construction would gi marked help in that direction.

Square gussets, of course, were an aesthetic abomin tion; but if one sold gussets they were cheaper th way!

The Victoria Station roof which Mr. Dailey mentione was designed in the London, Brighton and South Coa Railway Company's office by a Mr. Osbalddeston, a Mr. Gardner was in the Chief Engineer's office soon aft that time serving under him.

In reply to Mr. King's remarks concerning concre stresses, he reiterated his previous remarks, and he fe that there was no risk in increasing the stresses, provid that single intersection systems were not used, whi might permit cracking between the top and bottom bent up bars.

He was, of course, in agreement with Dr. Bateman, regard to the analytical approach in general, but explai ing his own view in that connection, he took as example the case of battened stanchions. If one spe the rest of one's life in trying to evolve a complete rational approach it would be wasting time; and be of n great value in practical design; and the number of yea that had been devoted to such work was already remar able. The analytical methods so far evolved could b better used as guides to simple rules which enginee could use; and rather than spend more time on furth analysis let us make typical battened stanchions and te them to arrive at comprehensive answers. He was for analytical enquiry, backed up in this way, but not f launching analysis into the engineering world at larg

here the majority did not fully understand it, and here it was in fact very seldom used.

The necessity for such enquiry being made by the big ganisations, so that the results would be useful to other ople, had been mentioned by Mr. King. If the general inciples were settled, the bigger consulting engineering ms and drawing offices could produce their own sheets data.

On Dr. Bateman's reference to anatomy, he said he had und that a lot of young designers simply did not under- and truss anatomy. Professor Fidler went into the atter in his book on bridge construction, one of the est books ever written; and every engineer should read and should keep the question of anatomy fully in mind hen designing.

He was very glad that Mr. Palmer had made reference shoe intersections. The intersecting of members at e shoe was very important and one often had to draw ttention to its neglect. If it were ignored as it often was in a rawing office, with a view to reducing costs, it was very ldom that the secondary effects were checked up. n the past there had been some failures in America sulting from this, in which the gussets had torn rough.

Mr. CREASY said that he was grateful to Mr. Andrews r having emphasised the point he himself had tried to ake; that it was extremely important to maintain erspective in regard to the problem of Economy.

As the late Professor C. M. Joad would have said, " it ll depends what you mean by 'Economy' ". This ord embraced so very many different interpretations ccording to the passion of the moment and he had eemed it more useful to write this third Section of the aper as referring to " comparative forms " of con- truction.

That brought him back to the point that in the solution f each problem as presented it was important to con- ider a broad series of illustrations from which a selection ould be made of that form of construction which best tted the immediate terms of reference. In these cir- umstances the Engineer's task was greatly assisted by eference to a Case Book setting out the potted experience f works constructed and investigations made, and the aper was intended as a first contribution to such a Case ook for which further examples would be gratefully eceived.

At the risk of drawing a shadow across the somewhat ver eulogistical approach to tubular construction which ad been in evidence during the discussion Mr. Creasy ointed out that one did not always necessarily effect big avings in cost by using tubes. Indeed, he had included n the paper a case in which a considerable extra cost had een incurred by the substitution of tubes for con- entional framework, thereby illustrating once again that ach problem must be solved according to the conditions eculiar to it. When these were favourable such as with ood repetition and big workshop demand the cost would e low; but at the other end of the scale the cost might ell be high where, for example, the detail was not suit- ble for the use of tubes, where there was not a workshop emand or where the tube availability was unsuitable. Each problem must be considered very critically and djusted according to the particular circumstances pplying at the time.

One general point arose from Mr. Mason's contribution n that the Author had tried to make it clear that the aper was deliberately drafted in the form of a Case ook with a view to inviting similar contributions and it as a somewhat negative approach to criticise the examples chosen because of those which had been missed out. For example; it was time of course that reinforced concrete beams could be constructed of the depth as steel beams but in the particular examples chosen the former were 1 ft. deeper than the steel; likewise in this particular case the increased cost of the building was offset by the reduced cost of steel reinforcement. No doubt other cases showed different results—it was the Author's purpose to invite these.

Concerning Fig. 3 the figures given for tonnage and cost were correct and from these the rate per ton could be deduced. Some rounding off had been made to the figures for the cost and weight of steel per sq. ft. of covered area but the Table as printed had been adjusted to set out the information in the form desired. To preserve the comparison the reinforced concrete con- struction is used only for the frame employing the same covering as for the other forms.

Mr. MORTON, dealing first with the President's question as to whether any advance had been made during the last 20 years or so in the methods of con- struction of breakwaters, said that, apart from the new types he had mentioned in the paper, he did not know of any. Unfortunately, breakwaters were not the sort of things that were made every day, and at any rate in this country he did not think that any had been commenced during the last 20 years or even more. A few had been constructed abroad, but he believed that generally they followed the traditional method and that not very much that was novel had been produced except for the Dutch design which was devised for the reclamation work on the Zuyder Zee, as described in the paper. He would be very interested to hear whether, during the storms of two or three months ago which had caused so much flooding in Holland, that type of construction had stood up to the waves better than the old form of dyke.

So far as the actual production of a breakwater type structure was concerned, he recalled that during the war the Admiralty had connected up several islands in the Orkneys by causeways to protect Scapa Flow, and whilst he did not say that they had introduced an entirely novel form of construction, they had introduced something new in building the causeway mounds which bear a close relation to breakwaters. They had a quarry in the Orkneys which yielded a poor quality stone which in the quarrying tended to break down into small pieces, and, as at the sites of the causeways swift currents ran continually, such small material as was quarried locally, if thrown into the stream, would have been swept away. Cement was in very short supply, and the idea was adopted of making bolsters of netting into which the small stones were filled; the bolsters were dropped into the stream, and this method proved to be very successful. The causeway slopes were protected by heavy concrete blocks. As could be imagined, a causeway built in the manner he had indicated, with the bolsters, was subject to considerable settlement. The bolsters gradually bound themselves together and, as they settled, the big blocks on the outside tended to slide down; and water passing through the mound tended to wash away the firm material through the interstices between the blocks. The means adopted to prevent the erosion and to stabilise the blocks was a method first practised by the Dutch, which was to deposit hot asphalte between the blocks. The mass of hot asphalte maintained its heat and plasticity for a sufficiently long time to penetrate into the interstices, binding the blocks together and pre- venting the scouring out of the smaller material behind.

Dealing with the reference in Mr. Mason's remarks to the use of prestressed piles and members in jetties, Mr.

Morton said he had purposely refrained from a full discussion of the general advantages and disadvantages of prestressed concrete in jetty work, but had dealt particularly with prestressed piles. The latter were now getting to be such stock commercial articles that they were produced at little, if any more, cost than the ordinary traditional concrete piles. At the same time, they required only about 20 per cent. of the amount of steel used in ordinary piles, and this saving in steel was one of the main reasons for mentioning prestressing in the paper, because, at the time it was written, steel was one of the structural materials which was in extremely short supply. All structural engineers would recall the difficulty there was at that time in getting deliveries.

As to the use of precast concrete bracings which one of the speakers had mentioned, Mr. Morton said he had used them, but considerable site work was involved in dropping the connections over the piles and grouting up the annular space which had to be provided to allow for t possibility of the pile not having been driven in exac the right spot. He believed the advantage lay in t type of thick slab construction, where there were mer vertical piles to support the load, together with rakers resist horizontal shock.

THE CHAIRMAN, in thanking the speakers in t discussion and the authors for having replied, said t paper was originally intended to introduce the questi of economy and to explain what was behind the Bullet It would be agreed, however, that the paper represent a valuable contribution in its own right and would be useful addition to the Institution's records. The pa had given rise to a very good discussion and an airing views, and even if the meeting had only been able emphasize, however incompletely, the problem economy, it would prove to have been worth while.

A Simple Theory of Suspension Bridges

Discussion on Professor A. G. Pugsley's Paper*

The PRESIDENT proposed a vote of thanks to Professor Pugsley for his valuable paper, and for the explanations he had given in presenting it.

Mr. H. H. LEYS (Associate-Member) submitted the following comments : " When I first perused Dr. Pugsley's paper, it seemed quite out of reach of an ordinary practitioner in short span suspension bridges. I therefore re-read the paper hoping for some easement from the voluminous calculations with which I have always had to surround real suspension bridges.

" With one sweep of the pen, the learned author abolishes all the effects of the extension of back stays, straight or loaded, the very real effect of suspended side spans on the deflection and distortion of the central span, the range of temperature, and half-a-dozen other matters that have to be selected and decided on before calculation can proceed.

" The dead load alone presents great difficulties in real life. The total horizontal pull, being about 3,750 tons, implies a cable weighing at least 350 lb. per foot. A stiffening girder of 80 sq. in. boom section with suitable web will need 650 lb. per foot. I have assumed in what follows, that each cable carries at leat two lines of traffic. With wind at only 20 lb./ft.^{2} the wind system will weigh at least 100 lb. and the floor framing with cross girders 50 or 60 feet long will need at least 380 lb. per foot, thus leaving less than 3,000 lb. per foot run for 25 sq. ft. of floor or about 30 lb. per sq. foot. I would deprecate using less than 45 lb., i.e., a factor of safety of 50 per cent. against the uplift of the 30 lb. wind envisaged in BS. 153 (1927).

" In the matter of live load it may be true that an allowance of 1,000 tons per cable is ample for assessing the cable area, but for deciding the loading and especially the partial loading, it is utterly inadequate.

" Putting aside the debatable *HB* loading (90 tons) of the *BS* now under draft, the average loading to be expected is the four-axled A.E.C. Foden lorry which operates on pneumatic tyres each carrying two tons. The maximum load per vehicle is therefore 32 tons on a loaded length of 24 feet.

" I have before me a graph showing the effect 22 ton mammoth A.E.C. lorries parked or halted at check and find from it that the intensity of loading two 10 ft. carriage-ways is 1.96 tons per ft. run, or loaded length of 40 feet, and 1.82 tons per ft. run, or loaded length of 100 feet or over. That is, the loadi to be taken for local bending in the application, is mu greater than that used in the design of the cable.

" Using Steinman's formulæ it is perfectly possible allow for these changes in ascertaining the load length. With this variant in mind where the ratio local loading for the girder to the general loading of t cable is about 3½ to 1 (a ratio that may well occur shorter spans), it seems to me that the effective load length will be a very much greater fraction of the sp than has been assumed in the paper.

" If we let the stiffening girder be, as the auth suggests, of uniform I of 5×10^6 in 4, and of dep 30 feet, then it will be the stiffest stiffening girder record. The Bronx Bridge girder is 11 feet deep 2300 ft. span, or 1/210 of the span ; that of the Gold Gate, 25 feet on 4500 feet, or 1/168 ; of the Geo Washington, 29 feet on 3500 feet or 1/120 but present example, 36 feet on 2500 or 1/84th of the sp

" It is therefore not surprising that even with i movable towers, the fraction of the live load (load over the whole span) carried by the girder amounts 18 per cent. I suggest that, with the addition of to relaxation and suspended side spans, the amount wo be nearer 28 per cent. and that would bring the gir towards the point where temperature would be material factor.

" If a graph is made (for a stiffening girder of const boom area and varying depth) of the relation of the lo carried $\frac{PL}{x}$ (set out vertically) to the moment inertia I (set out horizontally) and due regard is had the reversal at quarter point and stresses developed relation to an agreed compressive stress in *either* boo then it will be found (starting from zero share of live load carried by the girder and increasing the amo of inertia), that from $PL/1000$ to about 3 or $4 \times PL/10$ the load accepted by the girder rises very slowly a that for any small change of L the change of accep load is small. At about $4\,PL/1000$, however, the cu

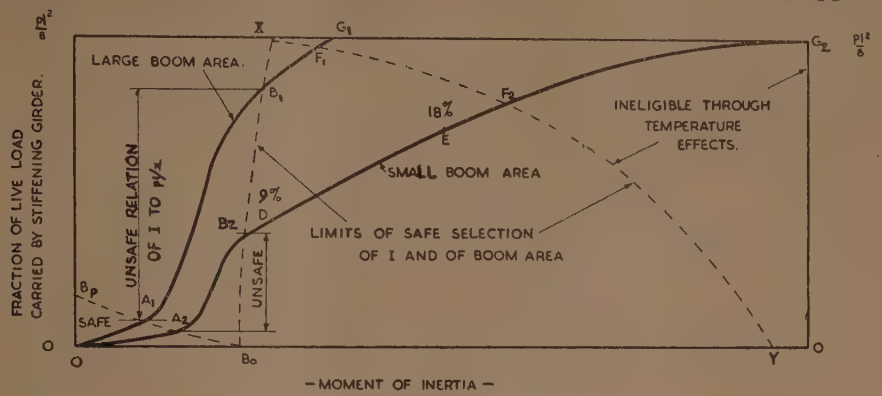
*Read before a meeting of the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, March 26th, 1953, Mr. E. Granter, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 3, pp. 75-82. (March, 1953.)

es sharply giving an area where a small change in *I* oduces a large change in the load carried and therefore the boom stress (an apparently unstable situation). eyond about 7 *PL*/1000 the curve flattens out and oceeds to its asymptote, at 125 *PL*/1000, near which int the temperature would become a major factor ace a very small extension in the cable would soon haust the limited capacity of the girder for deflection.

" I have allowed for an uplift of eleven tons per road track on a span of 280 feet, and if every condition of loading is considered I am certain it will be more on a light 2000 ft. span.

" Similarly, I am unable to conceive of the full and the dotted lines in Fig. 3 arising from the same formula.

" The full line shape suggests the elastic foundation formula and is only true or approximately true if the



Curves refer to constant boom area and varying depth.
OA_1, OA_2—George Washington bridge type (I very small).
A_1B_1, A_2B_2—position where δ (load)/δI is large and stress indeterminate.
D—Bottom limit of economy for small bridges.
E—Author's selection XY-incidence of temperature stresses.
F_1G_1, F_2G_2—Limits within which temperature is a major stress.
$B_p . B_o . B_2 . B_1 . X F_1 . F_2 . Y$—limits of area for safety.

" On plotting one or two curves for such cases by arying the boom area, it will be seen that, for any given oom area, there is a safe value of I, i.e., a safe girder epth which would give an economical weight of girder nd yet avoid the dangerous intermediate slope. If, of ourse, the bridge is so wide and so heavy as to be ndependent of wind effect and where the ratio of live o dead loading is very low as in the George Washington Bridge, it may be safe to use the flat part of the curve ear $PL = O$ in cable area.

" As regards suspenders, the author has ignored the act that the loading on his cable is of far greater quanity than any local load so that I suspect that when the uspender is fully loaded, i.e., when there are only a couple of lorries reacting in the same cross-section, then he displacement of the cable is so small that the deection of the girder over the span of two suspender anels has practically to be taken up by assuming the able immovable.

" I have done a little work on rail stresses with the elp of Indian railway papers 226 and 265 and IRE 1918, ut all these assume an infinite length of elastic foundaion, and I am frankly puzzled by section S4 and Fig. 2. The formulæ 9-12 predicate an infinite beam with nodes nd intersection (M-O) at pitches $\frac{x\pi}{4}$ $\frac{2\pi x}{4}$, etc. I lo not know if it is a coincidence that in Fig. 2, the lastic curve so drawn as to give M-O, at ratios equivalent o $\frac{x\pi}{4}$ and $x2\pi$ with x and 2π at the end of the span.

" The real loading would be an envelope of a series of hese curves depending on the loaded length. If, ccording to section S.5, the loaded length is of the nature of 500 ft. (then, owing to the concentration in eal life, it will be something less), then it is a physical ossibility that a very considerable negative moment would exist at the unloaded end.

second intersection of the curve with M-O occurs before the end of the span. The total curve of BM stressing is not only an envelope of the solid line for loading of all lengths and all agreed intensities but also that of the varying intensities for loads spreading from the centre. If these are plotted, it will be found that the reversal uplift BM is always a material proportion of the total BM which has to be resisted by the section modulus of the girder. I submit that, in what the author would call a short span (and I would call a span of normal length), the live load to dead load ratio is so high that no non-terminable equation could ever represent the facts, while, for the longer spans, it is the neglected condition at the end and the eighth or even tenth of the span range of loading that will produce the maximum range of bending moment. In addition, the crucial matter is not the total deflection or displacement, it is the radius of curvature that determines the stress.

" The essence of avoiding large, slow, dangerous vibrations in practice would surely be to create panels of slightly varying length, particularly at the centre, where the flatness of the cable makes the anode length of the girder similar to that of the cable, and the probability of synchronism high. It seems a minor point, but it should be possible by slightly altering the length of adjacent panels or the depth between flanges, or the shape of the boom of the girders (or all three) to produce a unit of either so high a frequency or of such high damping coefficient as to make the bridge stable."

Mr. O. A. KERENSKY first congratulated Professor Pugsley on his discovery of a new approach to the problem of suspension bridges. His paper was important for two reasons, first because it gave an easy understanding of the function of the stiffening truss, and secondly, because it gave a relatively quick approximation to stresses in the structure, with sufficient accuracy for a preliminary design.

The following data must be available before exact calculations can begin :

(1) Total dead load.
(2) Size of cable.
(3) Length and intensity of the live load which was likely to produce maximum moments at various points.
(4) Inertia of the stiffening girder.
(5) Chord Areas of the stiffening girder.

The last two items are the most difficult to guess, and the error in one affects the other. For example, if inertia were about right, but the area turned out to be too small, an increase in area increased the inertia, and that in turn increased the moment to be carried by the girder, thus requiring a further increase in the area, at the same time changing the gravity stiffness of the cable by the introduction of additional weight. An exact analysis by a team of experienced calculators took literally months to complete, and therefore it would easily be appreciated how important it was to have a quick approximate method for arriving at the required data with reasonable accuracy.

Professor Pugsley's method actually enabled one to calculate :

(1) Moments and shears in stiffening trusses.
(2) Length of load causing maximum moments.
(3) Pull in the cable.
(4) The load on hangers.

Of these, only the first two items were really useful, giving a very short cut in comparison with other known methods. The last two items were of no consequence ; the pull in the cable could easily be obtained from $\frac{wl^2}{8f}$, and the load on hangars was not important in the initial stages of design.

He was not clear whether Professor Pugsley claimed that the final design of an important bridge could be made by his elastic foundation method, but assume that he did not. In that connection Mr. Kerensk said that normally suspension bridges came into the own and became fully economic only for large span i.e., 1,000 ft. and upwards, there being no known upp limit, and for major and very expensive structures that kind the best analysis should be used, and yea of work in the office were fully justified by the resultin savings in the cost of such bridges.

The best exact method available to-day was that b C. D. Crosthwaite, who had refined and complete Professor Southwell's Relaxation method. At preser

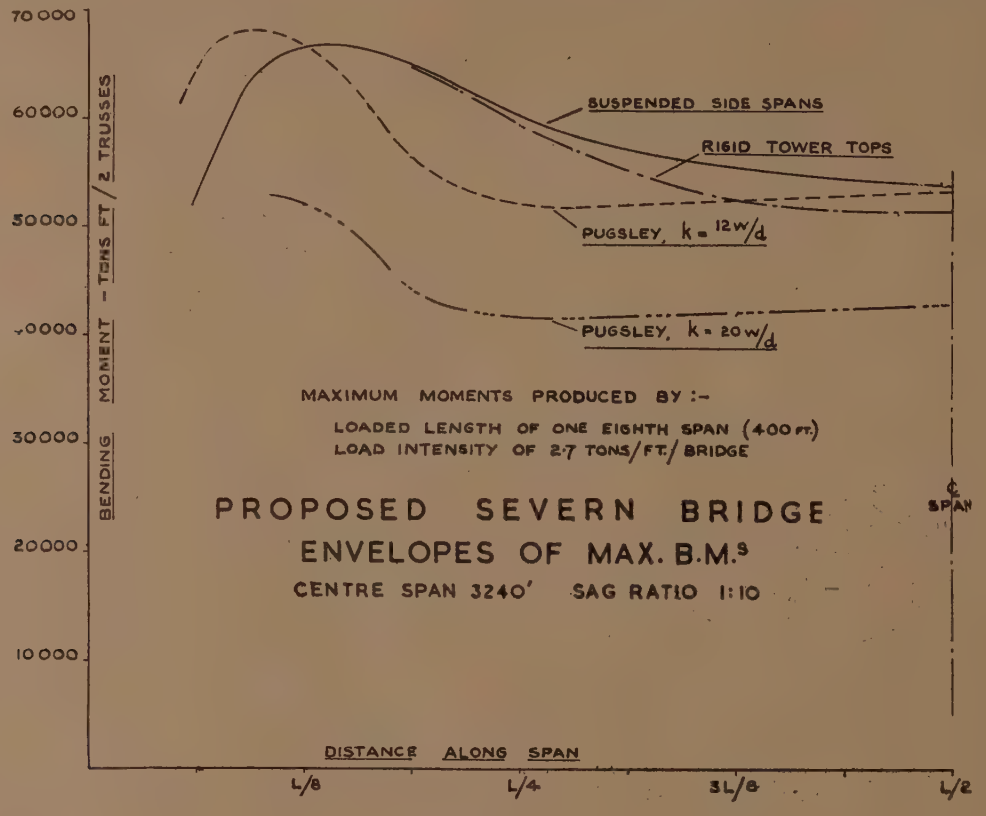


Fig. A

there was only one reasonably quick method of gettin the necessary approximate data, that by Hardesty ar Wessman. This semi-empirical method gave goo results, but took somewhat longer to apply than th one now produced by Professor Pugsley.

At this point Mr. Kerensky said that the classic theory of Rankine was absolutely useless for the desig of modern suspension bridges and should be left out text-books which attempted to deal with this subjec except for the purpose of exercises in structural in genuity ; it was misleading and confusing. Cou anyone imagine, he asked, that a 30-ft. truss uniformi distributed a point load over (say) 3,000 ft. span Anything might happen to a bridge designed on such misconception of facts. Perhaps we should soon se Professor Pugsley's method replacing Rankine's elementary text-books—not that the new method w elementary !

His firm had had an opportunity of checking Profess Pugsley's method, though not yet completely, again the more exact designs for the Severn and Otto Be bridges, and Mr. Wex would give a brief summary of th results of the checks.

As a minor point, Mr. Kerensky suggested that th presentation of the paper would be improved by elab orating the descriptions of the diagrams. Thus,

Fig. 1 the horizontal axis should be defined as "Deflection at any point due to load at that point," and in Fig. 2 the horizontal axis should be defined as "Bending moment for girder due to point load at quarter point."

Mr. S. P. WEX, before coming to his remarks on the checking of Professor Pugsley's method, first explained what he meant by the term "side span interaction." In the paper, he said, Professor Pugsley had assumed, for the purpose of a simplified analysis, that the cable was rigidly held at the tower tops. However, most bridges as constructed had either suspended or unsuspended side spans (straight backstays). In the case of a bridge with suspended side spans the effect of a live load on any part of the centre span was to deflect the tower tops towards the centre span, causing the centre span to sag, the side span cables to rise and as a result of this the side span trusses to hog. He then showed a line diagram of the proposed Severn bridge, having a centre span of 3,240 ft. between towers, and side spans of 1,000 ft. The diagram showed the general layout.

(*Fig. A*). Envelopes of maximum bending moment for the centre span of the proposed Severn Bridge, comparing values obtained by Professor Pugsley's approximate method and an "exact" method (Crosthwaite).

The Ministry of Transport type loading, varying in intensity with loaded length was used for this bridge.

It could be seen from the slide that :

(i) Bending moments in the centre span trusses were not much affected by replacement of suspended side spans by rigidly anchored tower tops.

(ii) The value of the foundation modulus, $k = 20w/d$, as proposed in the paper, did not give very good agreement with the exact solutions.

(iii) The value of $k = 12\ w/d$, chosen to produce the known maximum bending moment (about 67,000 t.f.) at the eighth point, gave values for the rest of the span which agreed fairly well with the exact values, the maximum error being around the quarter point.

It would be noted that from the quarter point to the half point the elastic foundation analogy gave practically constant bending moment, exhibiting the infinite beam characteristic stated in equation (11).

(*Fig. B*). Severn Bridge deflections. The curves showed the deflection at points along the span under the centre of a load, 400 ft. long, as it moved across the bridge.

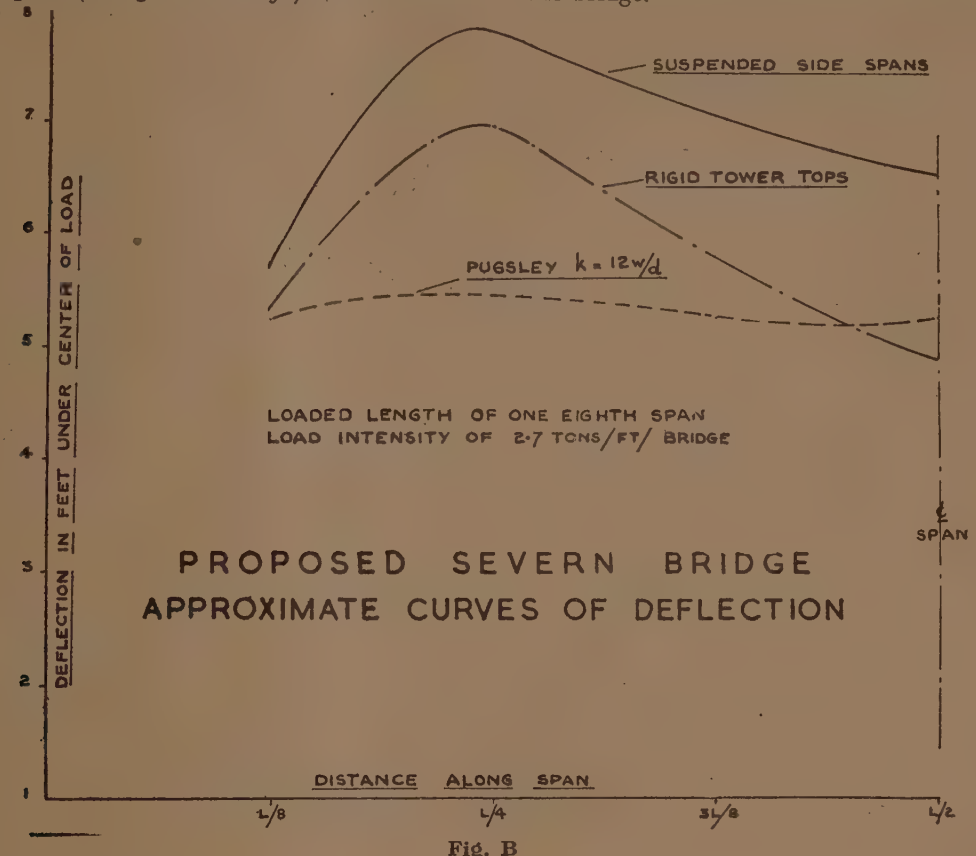


Fig. B

It could be seen that the effect of the relaxation of the tower top was to increase considerably the deflection under the load. Explaining what he believed to be the reason for the greater sensitivity of deflections, than bending moment, to side span interaction, Mr. Wex said that the answer lay in the length and flexibility of the trusses which could take up the extra deflections permitted by inward movement of the tower tops with only a very small change in truss curvature.

Professor Pugsley's method with a modified modulus gave good agreement for deflection at the $\frac{1}{8}$ point and at the $\frac{1}{2}$ point for the case assumed, that of anchored tower tops, but it was different at the $\frac{1}{4}$ point. There was a similarity between the two dotted curves shown on the slide and Fig. (1) in Professor Pugsley's paper.

From inspection of this figure one would expect maximum errors in both moments and deflections, due to the assumption of a mean uniform foundation modulus, to occur at about the quarter point.

He then showed an outline diagram of the Otto Beit Bridge in Rhodesia. The length of the centre span was 1,050 ft. The back stays were unloaded.

(*Fig. C*). Envelopes of maximum bending moment for the same bridge comparing values obtained by Professor Pugsley's method and the design curve (Moissief).

A loading curve of varying intensity was also used in the design of this bridge, but it was not as sharply varied as the Ministry of Transport curve.

In this case the modulus chosen to give good agreement with the design curve was $k = 10\ w/d$. This value might possibly have been deduced from the Severn Bridge value of $k = 12\ w/d$ (sag rates 1 : 10). The stiffness of a suspension bridge approximately varies inversely as the sag ratio so that for the Otto Beit Bridge sag ratio 1 : 8.4 one might expect that $k = 12\ w/d \times \frac{8.4}{10} = 10\ w/d$. For preliminary investigation of suspension bridge stiffening trusses the method of Hardesty and Wessman was useful and for an approximate method very accurate. It suffered, however, from three drawbacks.

(a) It did not give values of shear in the stiffening truss.

(b) It did not deal with loading curves of variable intensity and assumed that the critical loaded length was .4 of the span.

(c) It did not deal with a knife edge loading such as the Ministry of Transport required in addition to the variable distributed load.

Professor Pugsley's method was capable of dealing with these difficulties providing a suitable value of the foundation modulus was known. The values of suitable moduli might be obtained, for general use, from a statistical investigation of several existing bridges for which the relevant design figures could probably be easily obtained.

With regard to (b) above, using the formula on p. 79 of the paper with $k = 10\ w/d$ or $12\ w/d$ the critical loaded length of Otto Beit Bridge was estimated to be 4,200 inches ; it was in fact 4,400 inches. For Severn Bridge the paper gave 550 feet ; in fact it was 400 feet. One would expect the true critical loaded lengths to be shorter than those predicted by the paper because both bridges were designed for a variable loading curve. The degree of shortening would depend on the rate of change of loading intensity with loaded length of the loading curve.

Next, Mr. Wex queried the author's statement about the sensitivity of the truss moment to k. In equation (7) the moment in the truss was the result of a small difference between two large numbers, the simple beam bending moment and the moment for the foundation. As an example, using the figure of $k = 20\ \frac{w}{d}$ as postu-

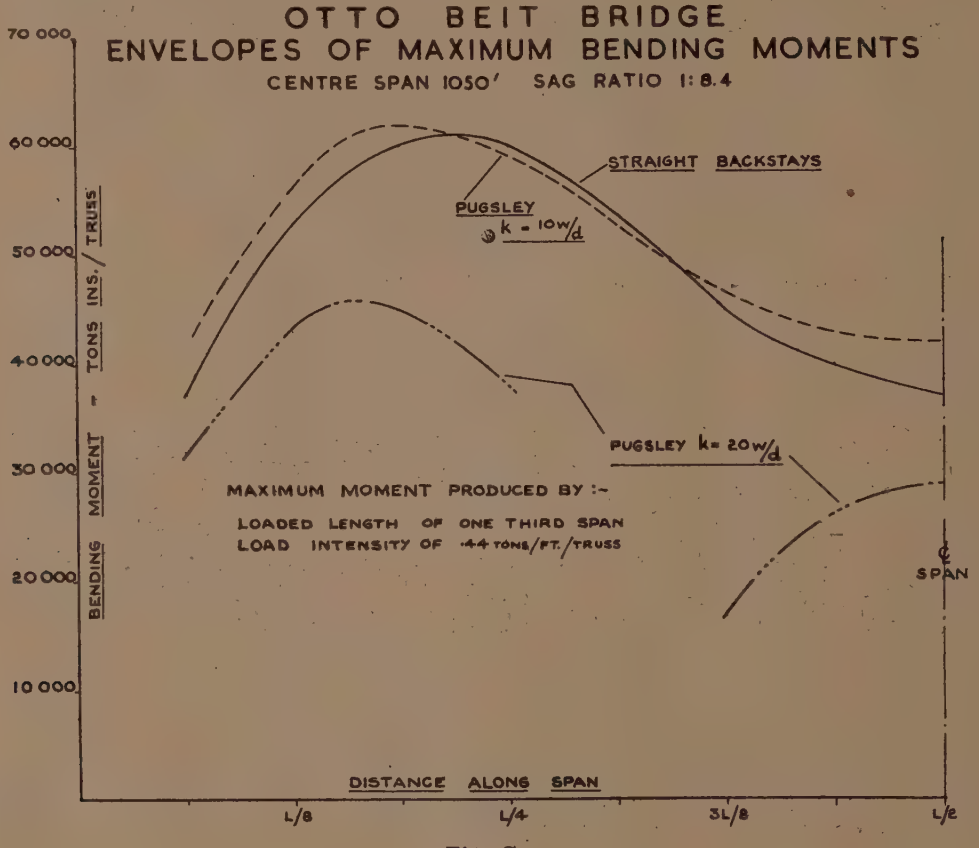


Fig. C

lated in the paper, he found that at the ¼ point of the Severn Bridge the exact moment taken in the cable was 542,000 tons/ft., and Professor Pugsley would say it was 558,000 tons/ft., a difference of only 3 per cent. But the moment in the truss, by the exact method, was 59,000 tons/ft., and the author's figure was 42,000 tons/ft., an under-estimate of 29 per cent. So that the moment in the truss was rather sensitive because of the fact that it was the difference between two large numbers.

An interesting point about the investigation described in the paper was that the modulus of $k = 12\ \frac{w}{d}$ continually cropped up ; Mr. Wex asked if an explanation might be as follows :

In Fig. 1 in the paper Professor Pugsley plotted the deflections at various points along a suspension cable when a single load was placed at that point. A mean deflection line was then assumed from which a value of foundation modulus was obtained.

If however one plotted the influence line for deflection of a gravity stiffened cable, say for the half point, it was immediately possible to see the effect on the half point deflection of loads elsewhere on the cable

n the case of Otto Beit Bridge such an influence line as plotted for a cable of sag ratio 1 : 8.4, and using the aded length of 4,400 inches symmetrically placed bout the centre line of the bridge the area under the nfluence line and thus the half point deflection of the able was obtained. This deflection was 204 in. for a ad intensity of .083 tons per inch (i.e., 1 ton per foot), that the foundation modulus was given by :

$$k = \frac{.083}{204} = .0004 \text{ tons per inch}^2.$$

hat value agreed with the one obtained from $k = 0 \cdot w/d$. Such agreement was of course somewhat rtuitous in view of the great simplification of the roblem.

Finally, Mr. Wex added his own congratulations to rofessor Pugsley for a most interesting paper and for aving put forward an approximate method of preiminary investigation of suspension bridge trusses hich he was sure was capable of a great deal of developnent, possibly on a statistical or semi-empirical basis.

Dr. E. H. Bateman (Member) commented that he as the only speaker so far who was not a designer of uspension bridges, and he had been greatly impressed ; f he had to design a suspension bridge he would be rateful for Professor Pugsley's theory, especially after earing the interpretation of it in the course of the resentation of the paper, which added greatly to the alue of the paper. He had read the paper carefully and ad tried to apply his mind to the method ; but Professor Pugsley had put it forward in a simple way which any tudent could understand. He hoped that that simple approach might be published in the Institution's Proceedings.

In considering Fig. 1 and Professor Pugsley's linear approximation, and then hearing that it did not quite it the facts when applied to bridges which had been completely analysed by all the modern methods, he wondered whether there might be a second stage in that approximation, because the curve of cable stiffness distribution looked like a mixture of the first and third harmonics.

Mr. E. Markland (Associate-Member) said it seemed hat the analogy of an elastic foundation was not exact. As Professor Pugsley had pointed out in the paper, an lastic foundation is taken to behave as a series of ndependent springs, so that a load on one of them did ot affect any other, whereas a load applied at one point f a cable not only depressed this point but raised other oints by amounts of the same order. It seemed that he discrepancy to which Mr. Wex had referred was ather more than could be accounted for by a slight nistake in evaluating the average cable stiffness, and he sked if this discrepancy might be due rather to the different behaviour of the two systems.

A value of 4.76 per cent. of the cable dip had been uoted as the mean deflection of a loaded point of the able when a load of 0.1 wL was applied at the point, ut Mr. Markland had not been able to arrive at this alue from Fig. 1. He asked, therefore, how Professor Pugsley had derived it.

Mr. R. A. Dorton said Professor Pugsley had given a nethod of analysis which had the advantage that it was asy to apply and illustrated clearly the relative contriutions of the cable and the truss to the stiffness of a uspension bridge. He hoped his comments would not e considered too critical, in the light of the author's mphasis on the approximate nature of the theory and he proposals that had been made for more suitable alues of k.

The author had compared his new method with a calculation by the flexibility coefficient method. The flexibility coefficient calculation for a concentrated load at the ¼ point was made by considering only six suspension rods, and it was Mr. Dorton's opinion that that under-estimated the true maximum moment by

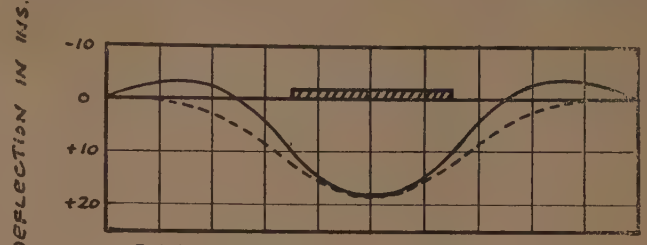


FIG.1. DEFLECTION DIAGRAM.

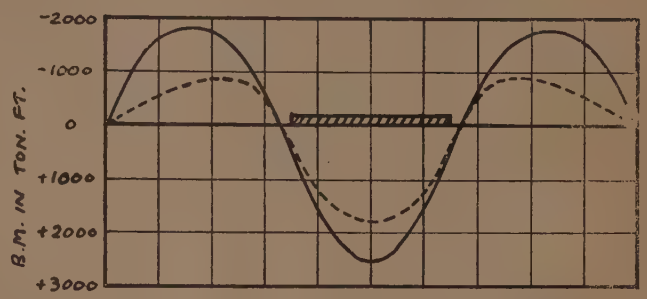


FIG.2. BENDING MOMENT DIAGRAM.

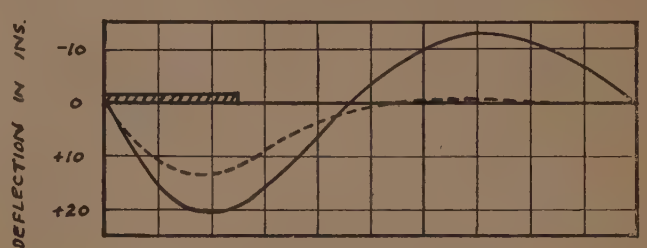


FIG.3. DEFLECTION DIAGRAM

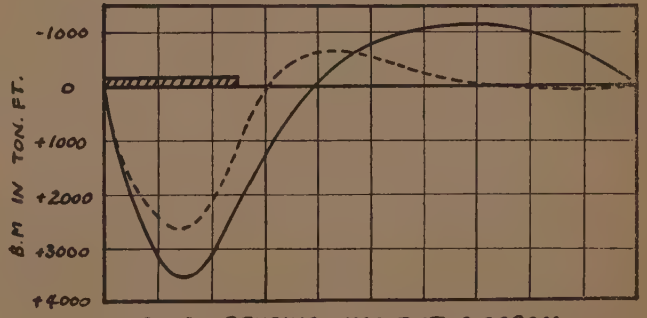


FIG.4 BENDING MOMENT DIAGRAM.

Dimensions Main Span = 1250 ft.
Cable sag = 125 ft.
Dead load per cable = 2500 lb./ft.
Live load per cable = 750 lb./ft.
EI (per truss) = 52.7 × 10^6 ton ft.^{2}.
Key Full lines—Timoshenko deflection theory.
Broken lines—Pugsley elastic foundation theory.

about 25 per cent. A comparison with a calculation by Timoshenko's deflection theory would give a more rigorous test of the accuracy of the elastic foundation analogy.

In the course of tests on a model representing a typical medium span highway suspension bridge he had carried out deflection theory calculations for the single span case for two different loadings, and had obtained close agreement between experimental and calculated values

of deflection and bending moment. For comparison he had also made an analysis by the elastic foundation method, using Professor Pugsley's suggested value for the modulus of k of $20\frac{w}{d}$.

Fig. 1 showed the deflected shape for a load covering the central portion of the span. The deflections at the centre agreed, as would be expected from the chosen value of cable stiffness. The fact that there was no upward deflection in the elastic foundation method illustrated the essential difference between a suspension cable and an elastic foundation, as had been pointed out previously by Mr. Markland.

In the corresponding bending moment diagram (Fig. 2) the maximum $+ve$ bending moment was about 25 per cent. below the deflection theory value, and the maximum $-ve$ bending moment was only about half the correct value.

The other case considered was for a load covering a quarter of the span from the left tower, Figs. 3 and 4. It was for a load near the towers that the assumption of uniform cable stiffness was most likely to be in error, and in Fig. 3 the downward deflections did not agree as they did with central loading. There was an upwards deflection of 13 in. by the deflection theory, but negligible upwards deflection by the elastic foundation method. The maximum $+ve$ moments again showed a difference of some 25 per cent.

By means of a suitable choice of value for k, the method appeared to be attractive for calculating positive moments and deflections. However, the fact that there was an appreciable upwards movement of a cable remote from its loading point meant that negative moments and deflections calculated by the elastic foundation analogy would be unreliable.

A knowledge of both maximum negative and positive moments was desirable for the correct proportioning of the chords in the truss, and the true deflected shape was needed to calculate the maximum grade change, which was a governing factor in the choice of a suitable stiffening truss.

Mr. W. Merchant (Associate-Member) said there were two problems, firstly that of selecting the average cable stiffness, and secondly of including for the effect that a load in one place caused deflections elsewhere in the cable. If the ordinary cable stiffness were used it corresponded to solving a structural problem only using the terms of the leading diagonal, a procedure which was a standard first approximation in the relaxation technique and gave good results only if the other influence coefficients were small. Even if the other influence coefficients were not small it should be possible to modify the cable stiffness to include for that effect and thus produce a theoretical justification for using a modified cable stiffness.

Professor Pugsley, replying to the discussion, said he was very glad indeed that his paper had provoked such valuable contributions from others working in the field ; naturally, that was one of the things he had hoped for.

Perhaps Mr. Ley was expecting him to run before he could walk. One realised, of course, that the ideal case of a simple cable with a girder underneath did not commonly arise in practice ; one must allow in practice for the side spans, live load, the extensibility of cables, and so on. But in trying to introduce a possible basis for a simpler method it was surely wise to start with the standard academic problem and then proceed to any necessary elaborations of it. It might not, indeed, be worth while to extend the simple theory too much, because what was wanted was an approximate and physically clear method ; if it were too elaborate it would defeat its purpose. So that whilst he had in mind on a future occasion trying to improve the method, one should not go too far for the sake of decreasing probable errors from, say, 10 per cent. to 1 or 2 per cent.

He had not put forward his examples for any purpose other than to demonstrate procedure ; they were not intended to represent practical cases or to form a close check on accuracy, but rather to see that things were in the right street and to demonstrate how relatively easy the method was to work if we could only find a satisfactory means, as had been discussed, of finding the value of the required foundation modulus k.

Mr. Ley had also mentioned that uniform live loads were somewhat academic. Although the theory did provide a mechanism for dealing with concentrated loads, probably a better mechanism than some existing theories, he doubted whether concentrated load cases were of much *general* design significance.

He was very much indebted to the speakers who had done so much work to illustrate the accuracies and inaccuracies of the method and to help towards arriving at a more suitable value for the foundation modulus. He agreed entirely with several speakers that, so far as negative bending moments and the upward deflections were concerned, no elastic foundation theory could hope to be very accurate, because an inherent feature of elastic foundation theory was a neglect of interactions between one part of the foundation structure and the other.

Several examples of extremely good agreement between the results of the theory and more accurate results had been given by Mr. Wex, and Professor Pugsley said it was of course fortunate that they had worked out as they did. But he regarded some of these extremely close comparisons as lucky shots. We could not expect an accuracy of within 2 per cent. consistently ; his attitude at the present stage was that he would be satisfied to be within 10 per cent., and he would not mind much even if occasional errors of 20 per cent. arose.

That brought him to Mr. Wex's suggested theory for the reason why the required foundation modulus was lower than he (Professor Pugsley) had expected ; why, in fact, the constant in equation (4) should be nearer 10 or 12 than 20. At the time of writing the paper, he was interested mainly in getting (4) into the right form. It was early yet to produce a simple theory of the sort under discussion and expect numerical agreement straight away. If we were now able to do what Mr. Wex wanted, we should be doing better than those who had been working for years on soil mechanics ; they had not yet produced a sound theory with which to relate foundation modulus and real test results.

With regard to Mr. Wex's tentative suggestion concerning the use of an influence line to explain the low k value, Professor Pugsley said he had tried something like it, but he did not feel satisfied with the soundness of the explanation when he had attempted to write it down. Perhaps Mr. Wex felt rather uncertain, too ; the interaction matter which was essentially missing in any simple foundation theory.

An alternative approach had occurred to him, but he had not dared to put it into the paper. He happened to be interested in the vibrations of suspension bridges, and had quoted some natural frequency results in the

aper. Equation (27) in the paper gave an expression or the natural frequency n'', in almost exact agreement with more accurate theory (equation (26)). This requency corresponded to a disturbance of the foundation of the small wave length. He had thought this esult not unreasonable, partly because the only comparative results he had quoted for bending moments had been for a concentrated load which also gave a hort disturbance. Design lengths of loading of the rder of 0.3 or 0.4 of the span naturally gave rise to onger disturbances, and one might expect that a onstant which gave accurate frequencies for modes with longer wave lengths might be more relevant. Thus we might be led to choose k, and therefore α in (20), such that (22) and (25) gave close results for n''; this required a value for k of 9.5. Perhaps by discussing the spread of the disturbance we might be able to arrive at why the constant should vary between one spread of load and another; but the evidence is not conclusive that this is the case.

Dr. Bateman had wondered whether, instead of taking a constant stiffness, we should take the first two terms of a heavier series to express the stiffness. No doubt one could do that, though the theory would be more troublesome; but Professor Pugsley said he was not trying to invent a new theory for elastic foundations. Maybe we could go back to the American deflection method—the differential equation method—and find that, with the foundation approach, we had effectively solved the equation by using a finite number of terms of particular form. Dr. Bateman's suggestion might in effect lead to this in better form; but by so doing we should probably have lost any physical notion of what we were doing.

It had been pointed out by Mr. Dorton that quite clearly the approximate method must get less accurate as we approached the end of a bridge, where there arose the effects of the local end conditions. Provided we were discussing lengths of loading of $\frac{1}{8}$ or $\frac{1}{4}$ of the span, the effects were not likely to be great.

He had already commented on the fundamental point raised by Mr. Merchant. A way of deciding upon k without complicating the method, by using the diagonal line of a flexibility table on the lines suggested by Mr. Merchant seemed feasible, and he thanked him for this line of thought.

Finally Professor Pugsley expressed his appreciation of the very interesting discussion of his paper; he was very glad to find that so many workers in both the design and research fields, were devoting attention to the subject. To twist Mr. Leys' references to Uncle Remus, if the paper had started a hare, then the discussion had gone a long way towards catching it.

The PRESIDENT expressed gratitude to Professor Pugsley for having replied in so able and interesting a manner to the many points raised in the discussion.

In a written addition to his reply to the discussion, Professor PUGSLEY said that his paper was prepared with the problems of modern long span suspension bridges in mind. As Mr. Kerensky had indicated, in most circumstances suspension bridges only became economical for spans of 1,000 feet and over. For the bridges of a few hundred feet span discussed by Mr. Leys, no doubt the more general features of the long span problem applied, but quantitative relationships, such as the stiffness ratio R of the paper, were more variable and he could not generalise on the accuracy of modern methods of anlysis—"exact" or approximate—in relation to the earlier short span suspension bridges. It was, of course, natural for many theories to be restricted to some regime set by scale; even Newton's Theory of Gravity needed modification on the astronomical scale!

Part of Mr. Leys' contribution was devoted to comments on the application of elastic foundation theory to cases where the beam and foundations were not of infinite length. Perhaps Mr. Leys had not used current extensions of the theory to cases of finite length and had so failed to note that results (5), (6), (7) and (8) were strictly accurate for such cases. Admittedly, these results involved Fourier series summations, but Professor Pugsley, with Mr. Kerensky and others who had used them, could assure Mr. Leys that these numerical summations were child's play compared with the more accurate calculations that were now commonly undertaken for long span suspension bridges. Mr. Leys was naturally anxious to see bridge natural frequencies in flexure (in all modes) put up. Unfortunately, as formulæ (24) and (25) showed, the lower natural frequencies of a suspension bridge were largely determined by the gravity constant g, and the dip d, of the bridge cable; increase of girder stiffness did, of course, increase these frequencies, but at a slower rate than given by the usual square root rule.

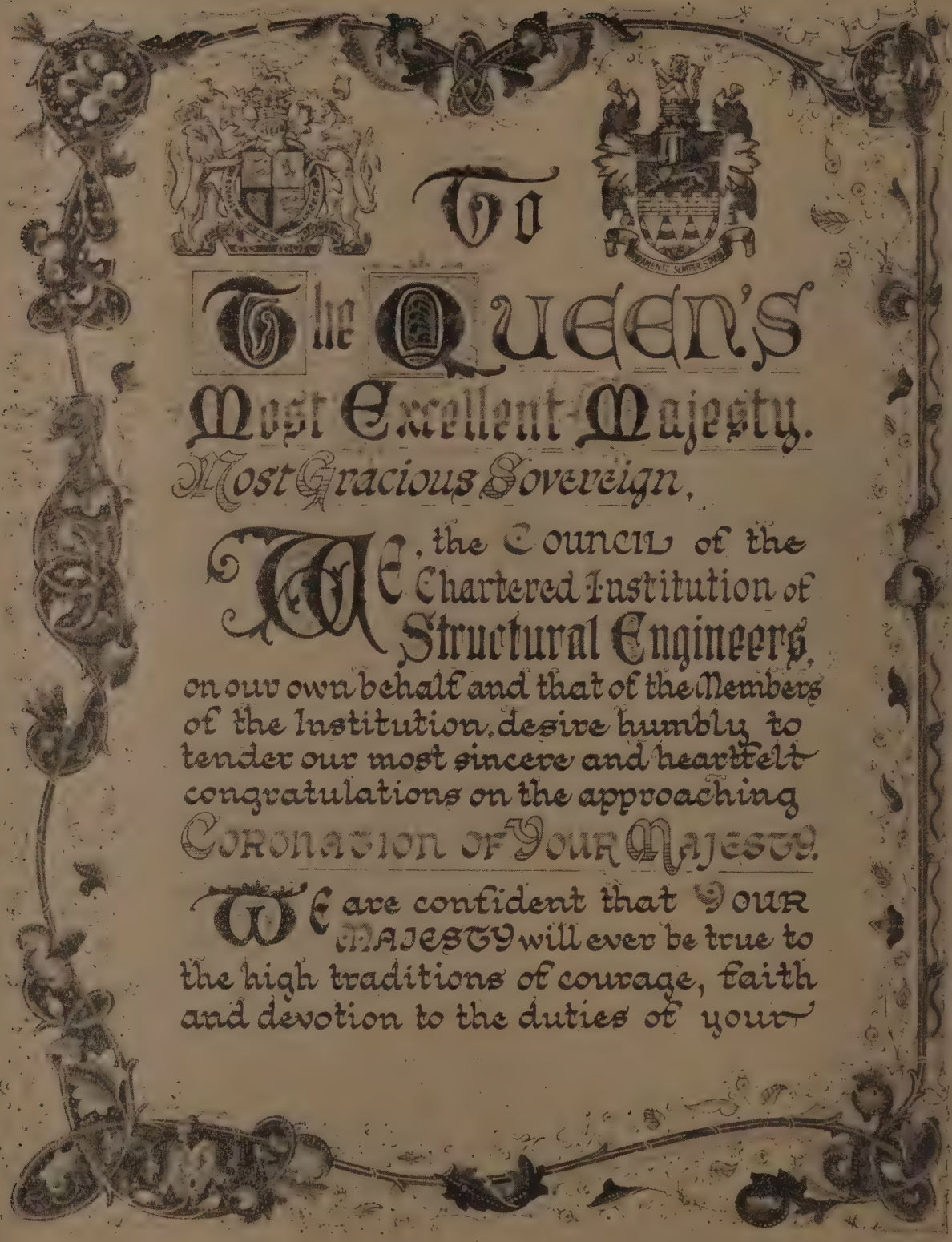
Professor Pugsley, when introducing the paper to the meeting, had made it clear that he regarded the elastic foundation analogy as providing an approximate tool for use in initial design. In that the method depended upon a physical understanding of the behaviour of a suspension bridge, he hoped, with Mr. Kerensky (though without perhaps calling for the complete dismissal of the time-honoured method of Rankine), that the analogy method would be of educational use.

Professor Pugsley was very interested in some of the details of Mr. Wex's results. The curves of his first diagram illustrated very nicely the effects of the particular conditions at the end of the bridge span; though not marked—amounting at the most to about 18 per cent. in the case illustrated—he wondered whether it would not be worth pursuing the matter by replotting the curves with different girder end support conditions. He had not expected the deflection results shown by Mr. Wex; it was usually easier to devise approximate ways of estimating deflections than of estimating local bending moments, yet the foundation analogy method seemed to show a tendency to contrary properties.

Mr. Wex had commented on the fact that equation (7) gave results in the form of differences between two large quantities and wondered if this did not mean that bending moments were more sensitive to the modulus k, than the author had supposed.

Professor Pugsley would point out that this numerical difference was an inconvenience rather fundamental; the physical fact noted in the paper that, for a *concentrated* load, the bending moment was inversely proportional to the 4th root of k, could be supplemented by studying equations (11) and (13) and the intervening theory. From this it was clear that for a short *uniform* load—and Mr. Wex's figures referred to this case—the bending moment must tend to vary inversely as the 2nd root of k, and this would approximately explain Mr. Wex's results.

In answer to Mr. Markland's inquiring about the origin of the figure of 4.76 per cent. used for the first rough estimation of k, this was based, with the curves of Fig. 1, upon previously published experimental results. The theory of reference 8 gave comparable values. In both cases, only small deflections were envisaged.



To

The QUEEN'S

Most Excellent Majesty.

Most Gracious Sovereign,

WE, the COUNCIL of the Chartered Institution of Structural Engineers, on our own behalf and that of the Members of the Institution, desire humbly to tender our most sincere and heartfelt congratulations on the approaching

CORONATION OF YOUR MAJESTY.

WE are confident that YOUR MAJESTY will ever be true to the high traditions of courage, faith and devotion to the duties of your

exalted office which were so ably maintained by YOUR MAJESTY'S late father King George the Sixth.

THE task which YOUR MAJESTY has been called upon to undertake is difficult and onerous, but the support of a free and loyal people on which YOUR MAJESTY may ever rely, will we venture to hope, ease the burden of that task.

WE pray that the blessing of Almighty God will continue to rest upon Your Royal Household and that YOUR MAJESTY may long be spared to reign over a united and loyal people.

FOR the Council of the Institution of Structural Engineers.

E. Grantor
President.

R. Emmerland
Secretary.

Soil Mechanics in Relation to Structural Engineering*

Written Discussion on Mr. P. L. Capper's Paper

From Mr. S. P. BANERJEE (Associate Member) : The writer appreciated the paper from the point of view that it summarised the factors which the Structural Engineers should keep in mind when dealing with problems on foundation design. On the question of choice of foundation, whether rigid or flexible, a design which keeps the balance of the elastic behaviours of the soil and the foundation structure, in keeping with the permissible relative settlements, would appear to be most satisfactory.

The author's reference to the " Soil line " method of design of Raft foundations was noted and the following remarks, which may be relevant to mention and prove useful to note, are added in this respect.

The writer finds that the method of solution is based on an assumption that the moments along the length of a beam are proportional to the deflections (pp. 24 and 87 of ref. 21). To obtain the maximum moment values, therefore, an attempt is made to find the maximum possible value of deflection " Y," even at the expense of involving a higher value of " q." This may be observed from eqn. (B) of p. 87 and Fig. 58 of the publication.

It is simple to realise that a reduction in the sum of the moments $M_a + M_b$ is possible due to the factor " q," since the reducing moment M_c is a function of q. The lesser the value of q, the lesser would be the consequent moments M_c and the higher the final moments M. It is apparent, therefore, that the least possible value of q should be aimed at for the higher possible moments in a beam and thus the factor C_{min} in eqn. (B), p. 87, requires to be replaced by C_{max}, which by solution gives a lesser value of Y. The relationship of proportionality between M and Y does not therefore hold good, proving the main assumption to be incorrect.

Consequently it is not difficult to prove that in the very first example in the publication (p. 56), the value of maximum moment obtained by the method is out by 40 per cent., erring on the wrong side. Also, without having the knowledge of the correct solution in a particular case, working between limits (pp. 23 and 28) may not cover the extreme cases. The writer is therefore doubtful about the reliability of the method the author suggested.

There are several theories on the subject available in our present-day knowledge and some of them are included in the following references :

(i) Beams on Elastic Foundation, by Hetenyi.
(ii) Trans. A.S.C.E., Vol. 116, 1951, p. 1083.
(iii) Int. Assn. for Bridge & Struct. Eng., 4th Congress, 1952, Prel. Publn., p. 371.
(iv) THE STRUCTURAL ENGINEER, August, 1952, p. 169.

All these theories make use, as is customary in theoretical analysis, of the coefficient " modulus of foundation or subgrade reaction," and it is not difficult to obtain the reasonable values from the sites. References may in this respect be made to the paper entitled " Field Tests Determine Foundation Modulus for Bayou Bodcau Dam," published in CIVIL ENGINEERING (U.S.A.) July, 1950, p. 24.

*Read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, February 26th, 1953, Mr. E. Granter, B.Sc. (Eng.), M.I.C.E., M.I.Struct. E. (President), in the Chair. Published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 2, pp. 47-61 (Feb. 1953). See also Discussion published in THE STRUCTURAL ENGINEER, Vol. XXXI, No. 7, pp. 190-6 (July, 1953).

Institution Notices and Proceedings

LOYAL ADDRESS

This issue includes a reproduction of the Loyal Address of the Institution to Her Majesty, in connection with which the President has received the following communication from the Home Secretary :—

Home Office,
Whitehall.
June 12th, 1953.

Sir,

I have had the honour to lay before The Queen the Loyal and Dutiful Address of the Council of the Chartered Institution of Structural Engineers on the occasion of Her Majesty's Coronation, and I have it in Command from The Queen to convey to you Her Majesty's warm thanks for the expressions of loyalty and devotion which it contains.

Her Majesty was graciously pleased to remark on the beauty of the Address.

I am,
Sir,
Your obedient Servant,
(signed) DAVID MAXWELL FYFE.

FORTHCOMING MEETINGS

The following meetings of the Institution will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, October 1st, 1953

Presidential Address, by Lt.-Col. R. F. Galbraith, M.C., R.E., B.Sc., M.I.Struct.E., A.M.I.C.E. 6 p.m.

Thursday, October 22nd, 1953

Ordinary General Meeting, 5.55 p.m., followed at 6 p.m. by a Joint Meeting with the British Iron and Steel Research Association and the Engineers' Group of the Iron and Steel Institute, when Mr. J. S. Terrington, B.Sc.(Eng.), A.M.I.C.E., and Dr. J. M. Hawkes, B.Sc., A.M.I.C.E. (Associate-Members), will give a paper on " Crane Gantry Girders for Steelworks."

Thursday, November 12th, 1953

Joint Meeting with the British Section of the Societe des Ingenieurs Civils de France, 6 p.m., when Monsieur L. P. Brice will give a paper on " Different Types of

Wharves and Maritime Structures recently executed in France."

Thursday, November 26th, 1953

Ordinary General Meeting, 5.55 p.m., followed at 6 p.m., by an Ordinary Meeting, when Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E. (Member of Council), and Mr. K. H. Best, B.Eng (Associate-Member), will give a paper on "The Reconstruction of a Soaking Pit Building."

Members wishing to bring guests to the Ordinary and Joint Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 5th and 6th (Graduateship), and January 7th and 8th, 1954 (Associate-Membership).

REPRESENTATION

The Council has made the following nominations of members to represent the Institution :—

City and Guilds of London. Institute—Insignia Award in Technology (Assessor)
Mr. J. Singleton-Green (Honorary Librarian).

Royal Technical College, Salford—Building Advisory Committee
Mr. W. Bates (Member)—re-appointment.

Royal Technical College, Salford—Engineering Advisory Committee
Mr. J. Kershaw, Jun. (Member)—re-appointment.

PROFESSIONAL CONDUCT

A Member of the Institution while practising as a consulting structural engineer has been reprimanded by the Council for soliciting professional work and informed that continuance of such solicitation will be considered by the Council as a serious offence.

FORMS OF CONTRACT

The Institution's Form of Contract, Form "A"—For the Engagement by the Principal of the Services of a Consulting Structural Engineer, is in the course of being revised and when such revision is completed, Form of Contract, Form "B"—For the Engagement by the Architect of the Services of a Consulting Structural Engineer, will be withdrawn.

By order of the Council,
R. F. MAITLAND, Secretary.

CONFERENCE ON WELDED STRUCTURES

Outline Programme

As announced in the August issue, a Conference will be held in London on November 23rd-27th, 1953, to review present knowledge and outstanding problems in the design, fabrication and erection of welded steel structures.

Sponsors

The organisation of the Conference is being undertaken by a Joint Committee appointed by the Ministry of Works, the Institution of Civil Engineers and the Institution of Structural Engineers with the support of the Admiralty, War Office, Ministry of Transport and Department of Scientific and Industrial Research.

The following bodies have agreed to be co-sponsors :—
Lloyds Register of Shipping.
The Royal Institute of British Architects.
The Institution of Mechanical Engineers.
The Institution of Engineers and Shipbuilders in Scotland.
The Institution of Naval Architects.
The Institution of Gas Engineers.
The Iron and Steel Institute.
The Institution of Electrical Engineers.
The Institution of Municipal Engineers.
The North-East Coast Institution of Engineers and Shipbuilders.
The Royal Incorporation of Architects in Scotland.
The Institute of Welding.
The British Welding Research Association.
The British Constructional Steelwork Association.
The British Iron and Steel Research Association.

Working Sessions

The working sessions of the Conference will be held at the Institution of Civil Engineers, Great George Street, London, S.W.1.

Eleven topics have been chosen for discussion and the procedure at the Conference will be to give the Author of a paper a short time to introduce his paper, leaving the rest of the session free for general discussion. The Author will be allowed to reply briefly to the points raised at the end of the session.

The following authors have been invited by the Organising Committee, and have kindly agreed to present papers on the subjects shown. No general invitations for the submission of papers is being made.

Paper

No. 1 Mr. W. A. Mitchell, M.I.Struct.E.
Design of multi-storey framed buildings.

2 Mr. D. C. C. Dixon, B.A.
Problems of fabrication and erection of multi-storey framed buildings.

3 Mr. F. C. Cocks, B.A.
General review of welding in ship-building.

4 Mr. F. J. Walker, A.M.I.C.E., and Mr. F. W. Sully, M.I.C.E.
Docks gates and caissons.

5 Mr. M. Noone, M.I.Mech.E.
Standing oil tanks and gas holders.

6 Mr. W. S. Atkins, B.Sc., M.I.C.E.
Design of single-storey framed buildings and portal frames.

7 Mr. R. W. Schofield, M.I.Struct.E.
Fabrication and erection of single-storey framed buildings and portal frames.

8 Mr. S. M. Reisser, B.Sc., M.I.Struct.E., A.M.I.C.E.
Design, fabrication and erection of braced roof structures.

9 Mr. G. Roberts, B.Sc., M.I.C.E., and Mr. O. A. Kerensky, B.Sc., M.I.C.E., M.I.Struct.E.
Design, fabrication and erection of plate girder bridges.

10 Mr. E. M. Lewis, A.C.G.I.
Design, fabrication and erection of trussed girder bridges.

11 Mr. T. Bedford, M.I.C.E., M.I.Struct.E., M.I.Mech.E.
Tubular construction.

Advance copies of each of the papers will be issued to all members before the Conference.

After the Conference, the papers of the discussion will be published in book form as a permanent record of the proceedings.

Membership

The Conference is open to all interested in the subject, who register as members. Applications for membership should be made on the requisite form, which may be obtained from the Secretary of the Institution of Structural Engineers.

Lunch

Buffet lunches, which will be served at the Institution of Civil Engineers, will permit members to meet and discuss problems informally.

Fees

A registration fee of £2 10s. will entitle members to receive full details of the Conference Programme ; to take part in all the activities of the Conference ; and to receive one copy of each paper and one copy of the Proceedings. The remittance must be sent with the application form to the Joint Honorary Secretaries before October 1st, 1953.

Provisional Time-Table

Monday, November 23rd

10 a.m. The Secretariat Office will be open to receive enquiries.
2.30 p.m. Formal opening (by a Minister of the Crown).
3 pm.-5 p.m. First Working Session—Multi-storey Buildings. (Papers Nos. 1 and 2.)

Tuesday, November 24th

10 a.m.-1 p.m. Second Working Session—Shipbuilding. (Paper No. 3.)
1 p.m. Buffet Lunch.
2.30 p.m.-5.30 p.m. Third Working Session—Dock gates and caissons. (Paper No. 4.)
Standing oil tanks and gas-holders. (Paper No. 5.)

Wednesday, November 25th

10 a.m.-1 p.m. Fourth Working Session—Single-storey buildings. (Papers Nos. 6 and 7.)
1 p.m. Buffet Lunch.
2.30 p.m.-5.30 p.m. Fifth Working Session—Braced roof structures. (Paper No. 8.)

Thursday, November 26th

10 a.m.-1 p.m. Sixth Working Session—Bridges. (Papers Nos. 9 and 10.)
1 p.m. Buffet Lunch.
2.30 p.m. Seventh Working Session—Tubular construction. (Paper No. 11.)
Chairman's closing remarks.

Information

The Joint Honorary Secretaries for the Conference are :—

Mr. E. Graham Clark, Secretary of the Institution of Civil Engineers, and

Major R. F. Maitland, Secretary of the Institution of Structural Engineers.

Application forms may be obtained on request from the Secretary of the Institution of Structural Engineers.

The completed application forms and all requests for information should be sent to :—

Joint Honorary Secretaries,
Conference on Welded Structures,
Institution of Civil Engineers,
Great George Street,
London, S.W.1.

SYMPOSIUM ON TESTING AND STRESS GRADIN OF TIMBER

A Symposium on Testing and Stress Grading Timber is to be held on September 22nd, 1953, at th Institution of Civil Engineers, Great George Stree S.W.1, when the following four papers will be presented

10 a.m. "Methods of Test of Small Clear Specimens, by F. H. Armstrong (Forest Produc Research Laboratory).

11.30 a.m. "The Testing of Structural Components an Structural Units," by J. G. Sunbury (Fore Products Research Laboratory).

2 p.m. "The Mechanical Testing of Sawed Timbe and Application of Data to Stress Grading, by W. J. Smith (Forest Products Labora tory, Canada).

3.30 p.m. "Strength Studies of Timber in the Develop ment of Structural Timber Grades" (Fore Products Laboratory, U.S.A.).

The symposium is being sponsored by the Incorporate Association of Architects and Surveyors, 75, Eato Place, London, S.W.1. Advance copies of each of th four papers may be obtained from the Associatio price 1s. 6d. The Secretary of the Association would b grateful for an intimation from members who hope t attend.

EXAMINATIONS

Preparation for the Examinations of the Institu tion by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membe ship may be able to attend a technical college ; thes notes are intended to guide him in choosing the mos suitable instruction.

Preparation for the Graduateship Examinatio

Technical Colleges offer :

(a) Full-time courses for degrees of Higher Nationa Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Highe National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diplom complying with Appendix II, Section V, of the Regula tions Governing Admission to Membership, the candidat will be exempted from the Graduateship Examinatio

Alternatively, he may study subjects selected from th available courses and sit the Graduateship Examinatio At technical colleges courses are usually available i Building Science or Engineering Science, Strength o Materials, Theory of Structures and Surveying, bu students are not normally allowed to select subjects fro National Diploma or Certificate courses unless they ca show evidence of sound training in more elementar studies. The advice of the College Authorities shoul be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time course in Structural Engineering which cover the syllabus of th Associate-Membership Examination. At other college the candidate must rely on Higher National Certificat courses or on advanced courses in Building, Civil En gineering or Municipal Engineering ; these cover onl

art of the requirements for the Associate-Membership xamination.

Colleges in List " A " provide at least two years of istruction in Theory of Structures and in Structural ngineering Design and Drawing up to Associate-embership standard. They also give instruction in tructural Specifications, Quantities and Estimates.

IST " A "

Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Salford, Royal Technical College.
South-West Essex Technical College, Walthamstow, E.17.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of tructures from which the student may reach Associate-lembership standard, but instruction in Structural ngineering Design and Drawing and in Structural pecifications, Quantities and Estimates is not usually o complete.

IST " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Plymouth and Devonport Technical College.
Preston, Herris Institute.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.
Students are advised to take the organised courses in tructural Engineering where these are available.

DRURY MEDAL AWARD

The fourth competition for the above award will take lace in 1953. The subject is the design of the structure f a new factory building. The material of construction s entirely at the choice of the competitor. The compe-ition has been designed to encourage ingenuity of tructural arrangement. Economy in the use of steel is n important feature of this year's competition.

Graduates and Students of the Institution who wish o compete are invited to apply for full details to the ecretary ; envelopes to be marked in the top left-hand orner, " Drury Medal Award."

The closing date for the competition is October 1st, 953.

The general conditions of the competition are as follows :—

1. The competition shall be for Graduates and Students of the Institution of not more than 25 years of age.

2. The subject of the competition shall be a design of a structural character, that is to say primarily structural design, not planning.

3. The subject of design and conditions shall be prepared and issued biennially by a group of five members appointed by the Council.

4. The Literature Committee shall appoint a Jury of not less than five to examine the works submitted and to interview candidates, if found necessary.

5. In order to show that the work submitted is solely the work of the competitor, the documents submitted shall be countersigned by a corporate member of the Institution or, failing this, shall be accompanied by a declaration on a prescribed form signed by the candidate in the presence of a Justice of the Peace or a Commissioner for Oaths.

RESEARCH AWARDS

The Council has instituted a Research Prize Fund, from which awards may be made annually to the author or joint authors of papers describing original research which they have carried out. Research awards may be made for papers read at Headquarters or in the Branches and published in the Journal, or for papers published in the Journal only without being read at an open meeting.

The assessment for such awards will be made annually, but awards will be made only to the contributors of such papers as reach a standard judged by the Literature Committee to be satisfactory.

Work submitted under this scheme must be original and may include any of the following :—

(a) investigations of an experimental or analytical character ;
(b) studies of historical or statistical records ;
(c) improvements in principles or methods of construction ;
(d) research into methods of structural engineering and building, the nature and use of plant and the organisation of engineering work ;
(e) any related or combined studies which are deemed by the Literature Committee to be of a research character.

In cases where the research work described in the paper was not the work of one individual, the names of all the collaborators should be given in the paper.

Awards may take any or all of the following forms :— A research medal ; a diploma ; a money prize.

Application for consideration for a research award must be made to the Secretary of the Institution, and in preparing papers for reproduction in the Journal, authors must comply with the conditions laid down for all such contributions. Particulars of these conditions may be obtained from the Secretary.

In judging research papers, the following factors will be considered :—

(a) the nature of the subject and its conclusions ;
(b) the value of the paper in advancing the science and art of structural engineering ;
(c) the standard of preparation and orderly arrangement of the subject-matter.

Research papers will also be eligible for adjudication for the Institution Medals if they comply with the Regulations governing those awards.

The closing date for the receipt of applications in respect of papers published in the Journal between October, 1952, and September, 1953, is October 1st, 1953.

LONDON GRADUATES' AND STUDENTS' SECTION

A visit to a site to see a prestressed concrete building in the course of erection is being arranged in October and details will be announced in due course.

A meeting of the Section will be held at 11, Upper Belgrave Street, London, S.W.1, on Tuesday, November 10th, 1953, when Mr. J. F. S. Pryke will give a lecture on Underpinning.

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings will be held in October :—

Wednesday, October 7th, 1953

Joint Meeting with the Institute of Welding, Manchester and District Branch, 7.15 p.m. Mr. S. M. Reisser, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., on "Drawing Office Procedure for Welded Designs."

Thursday, October 29th, 1953

At the University of Manchester, 6.30 p.m. Chairman's Address by Professor J. A. L. Matheson, M.B.E., M.Sc., Ph.D., M.I.C.E., M.I.Struct.E., followed by films entitled "750 Feet Television Mast at Sutton Coldfield," and "Lali Bridge." The President and the Secretary of the Institution will attend the meeting.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford.

MIDLAND COUNTIES BRANCH

The Annual Dinner of the Branch will be held at the Botanical Gardens, Birmingham, on Saturday, October 10th, 1953.

The opening meeting of the Session will be held on Friday, October 23rd, at the James Watt Memorial Institute, Birmingham, at 6 p.m., when the Chairman's Address will be given by Mr. G. E. Marsden, M.I.Struct.E.

The President and the Secretary of the Institution will be present on both occasions.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

A meeting of the Section will be held on Friday, October 30th, at the Birmingham Civic Centre, at 6.30 p.m., when a member of the staff of British Insulated Callenders' Construction Company, Ltd., will give a paper on "The Design and Erection of B.B.C. Television Masts."

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The opening meeting of the Session will be held at the Neville Hall, Newcastle, on Wednesday, October 7th, at 6.30 p.m., when Mr. T. H. Bryce, M.I.C.E., M.I.Struct.E., will be installed as Chairman and will give his Address on the "History of Reinforced Concrete." The President and the Secretary of the Institution will attend the meeting.

A meeting will be held on Tuesday, October 13th, at the Cleveland Scientific and Technical Institution, Middlesbrough, at 6.30 p.m., when Mr. Bryce will repeat his Address.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The opening meeting of the Session will be held at the College of Technology, Belfast, on Tuesday, October 6th at 6.45 p.m., when the Chairman's Address will be given by Mr. J. C. Malcomson, B.Sc., M.I.Struct.E.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The opening meeting of the Session will be held on Monday, October 26th, when the Chairman's Address will be given by Mr. R. H. Sharpe, M.I.Struct.E. The President and the Secretary of the Institution will attend the meeting.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., 33 Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held at Plymouth, Saturday, November 7th, and will be attended by the President and the Secretary of the Institution.

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay.

C. J. Woodrow, 23, Torland Road, Hartley, Plymouth.

WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held at Bristol on Friday, October 2nd, and will be attended by the President and the Secretary of the Institution.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 2 Southdown Road, Westbury-on-Trym, Bristol, 9.

WALES AND MONMOUTHSHIRE BRANCH

The opening meeting of the Session will be held at the South Wales Institute of Engineers, Cardiff, on Wednesday, October 28th, when Dr. A. A. Fordham M.I.Struct.E., A.M.I.C.E., will give the Chairman's Address: "Research Work in Soil Mechanics" at the University College of Swansea. The President and the Secretary of the Institution will attend the meeting.

The Address will be repeated at Swansea on Wednesday, November 4th, and at Colwyn Bay on Saturday, November 7th.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

YORKSHIRE BRANCH

The opening meeting of the Session will be held at the Great Northern Hotel, Leeds, on Wednesday, October 14th, at 6.30 p.m., when the Chairman's Address will be given by Mr. John Dossor, M.I.C.E., M.I.Struct.E., A.M.I.Mech.E. The President and the Secretary of the Institution will attend the meeting.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111. Ext. 257.

Natal Section, Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 1692, Cape Town.

The President, 1953-1954

The Institution wel-mes as its President for ie Session 1953-54 a ondon Scot, Lt.-Colonel . F. Galbraith, M.C., .Sc.(Eng.), A.M.I.C.E., ho takes office on Octo-er 1st.

Robert Forbes Galbraith as born in London and as educated at St. Dun-an's College, Catford, and ie London County Coun-l School of Building, rixton. His career was terrupted almost at its mmencement by the out-eak of war, and in 1915, the age of 18, he joined ie Inns of Court Regi-ent. He served with istinction in the Regi-ent and in the Machine un Corps until 1919, and as awarded the Military ross.

On demobilisation he ecame an external stu-ent of London University nd took the degree of .Sc.(Engineering). In 923 he became Director nd Engineer to Messrs. albraith Brothers, Ltd., nd was responsible for ie erection of numerous uildings in London and lsewhere, including Wim-ledon Town Hall, the Geological Museum, South ensington, Gerrard Telephone Exchange and Palmerston ouse, Bishopsgate.

Lt.-Col. R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E.

Twenty years after his return to civil life, his career as again interrupted by a world war, and early in 1940 e joined the Corps of Royal Engineers, in which he rved until 1945, first in a Construction Company and hen on the Staff. He was demobilised in 1945 with the ank of Lieutenant-Colonel.

In 1950, Colonel Galbraith commenced private ractice as a Consulting Engineer, and has specialised n building structures, including the six-storey Plashet ounty Secondary School for Girls at East Ham.

He is a member of the Association of Consulting ngineers and an Associate-Member of the Institution f Civil Engineers.

Colonel Galbraith's connection with the Institution f Structural Engineers began in 1915 when he was lected a Student of the then Concrete Institute. He ecame an Associate-Member in 1920 and a full Member f the Institution of Structural Engineers in 1929, and ince 1925 he has taken an active part in the work of ts scientific Committees.

In 1929, he was Vice-Chairman of the Sectional ommittee on Brickwork and Masonry, and served also n the Sectional Committee on Concrete; his member-hip of the Science Committee continued until 1939. In 932, he was Chairman of the Committee on Welded Structures, the findings of which were published in 1934 in the Institution's Report on the Treatment of Welded Structures by the Metallic Arc Process. Colonel Galbraith was elected to the Council in 1934 and served until 1937. During that period he also took part in the work of the Building Legislation Committee and the Sub-Committees on Steelwork Connections, Bearing Pressures and Soil Pressures.

He again became a Member of the Council in 1947 and was elected a Vice-President in 1949. Since then, he has served on the Finance and General Purposes (as Chairman in 1952-53), the Bye-Laws Committee, and Membership Committee.

Colonel Galbraith has represented the Institution on the Architects' Registration Council of the United Kingdom, the London County Council District Surveyors Examination Board, the Tribunal of Appeal in connection with the London Building Acts (Amendment) Act, 1939, and other official committees.

During his forty years' association with the Institution Colonel Galbraith has given unsparingly of his time and energy to further the aims and objects for which the Institution was founded. The Institution of Structural Engineers has itself been in existence for less than half-a-century and during that time has achieved an eminent position among chartered bodies. It has been a period when tremendous advances have been made in the development of the science of structural engineering and it is mainly through the untiring efforts of members who, like the President for the coming Session, have given so much time in serving upon its scientific committees, that such remarkable progress has been made in such a short time.

Colonel Galbraith is well-known to London members and he will undoubtedly make a host of new friends throughout the country during his term of office. Those who meet him for the first time will appreciate to the full his kindly personality, and the value of his opinion upon professional matters, based as they are upon experience gained both in the civilian and military fields, in which spheres the human relationship counts for so much.

Members in all classes of the Institution will look forward to a year of further progress under his Presidency and those who are responsible for conducting the work of the Institution will welcome the opportunity of serving under him.

Crane Gantry Girders for Steelworks*

By J. S. Terrington, B.Sc.(Eng.), A.M.I.C.E., A.M.I.Struct.E., A.C.G.I. and J. M. Hawkes, B.Sc., Ph.D., A.M.I.C.E., A.M.I.Struct.E.

Notation

A	=	cross-sectional area.
b	=	width of flange plate.
b_1	=	width of "outstand."
d	=	unsupported depth of web.
d_r	=	depth of rail.
d_1	=	overall depth of section.
d_2	=	depth back to back of angles.
d_e	=	effective depth.
D	=	flexural rigidity of a plate.
E	=	modulus of elasticity of steel in tension.
f_{b1}	=	maximum allowable compressive fibre stress in bending.
f_{b2}	=	permissible maximum fibre stress to avoid wrinkling of an unstiffened edge between stiffeners.
f_{b3}	=	permissible maximum fibre stress to avoid angular distortion of the compression flange between stiffeners.
f_{cr}	=	critical maximum compressive stress in web.
η	=	efficiency of section against lateral buckling.
η_c	=	modulus factor in compression.
η_s	=	modulus factor in shear.
I_s	=	moment of inertia of stiffener about the centre of the web.
I_{xx}	=	moment of inertia of section about axis X—X.
I_{yy}	=	moment of inertia of section about axis Y—Y.
K_1	=	bending stress factor as B.S.449, 1948, clause 19c.
k_{xx}	=	radius of gyration about axis $X - X$.
k_{yy}	=	radius of gyration about axis $Y - Y$.
L	=	span of girder.
M	=	bending moment.
q	=	stiffener factor.
μ	=	Poisson's ratio.
σ	=	percentage coefficient for end fixity.
s	=	stiffener spacing.
s_f	=	shape factor.
σ	=	direct stress in web.
t_1	=	flange thickness.
t_2	=	web thickness.
τ	=	shear stress in web.
τ_{cr}	=	critical shear stress in web.
W_E	=	total equivalent uniformly distributed load.
w	=	weight of girder per unit length.
Z	=	modulus of section.

NOTE : The notation generally follows B.S.560 : 1934, including the use of k for the radius of gyration. The current editions of B.S.449 and B.S. Code of Practice 113 (1948) have however used τ instead of k.

Summary

Crane gantry girders are particularly important in steelworks because production relies on the electric overhead travelling cranes, and experience shows that there is scope for improvement in the girder design. An endeavour has accordingly been made in this paper to show how improved design and resulting economy can be achieved by closer adherence to fundamental theory and to the best practice.

A single web type plate girder is shown to be preferable to other types and a formulæ for a girder of least weight is given. For the flanges, a new development of the strain energy equation is put forward in graphical form for use in practice and it is shown that for the stockier sections, the maximum permissible stress can be used over a greater range of slenderness ratios, and for the more slender sections over a more restricted range, than is suggested by B.S.449. By means of further development, a graph is produced which enables given sections to be examined for permissible stress and other graphs which enable suitable sections to be selected directly. The best web thickness is attained when the web panel reaches the yield point at the same time as it commences to buckle ; and formulæ are given for the various conditions which occur along the girder, namely pure shear, pure bending and combined bending and shear ; local concentration of stress is also taken into account. It is shown that the optimum web thickness differs from the value given in existing specifications. These formulæ are also adapted to give the necessary stiffener spacing and a special method has been devised to enable the equations to be quickly applied. Stiffeners must be sufficiently strong to prevent nodal lines from forming across them and this is defined by an equation which indicates that stiffeners in many existing girders are heavier than necessary. Details for end connections to permit rotation of the bearings and resistance to lateral and longitudinal loading and temperature variations are discussed. Camber, it is considered, is beneficial but may give rise to high fabrication costs. The actual loading exerted by the cranes, is shown to be a complex problem which depends on the relative masses of the lifted loads, the cranes and the girder and needs further consideration, but values which, from experience, are considered suitable, are given.

To sum up, the study presented in this paper, emphasises the best criteria for design and from this consideration of basic design theory, it is shown that fundamental design can be adhered to more closely with resulting economy particularly in the larger girders.

Introduction

Melting Shops are some of the most important buildings in steelworks and are also one of the heaviest forms of single storeyed steel-framed buildings. After examining these and making recommendations for improved design(1), it was considered that parts of the structure should be considered in more detail. Of these, the crane gantry girders are probably most closely related to production, since the electric overhead travelling cranes are in use throughout the day and night, and invariably every day of the week, so it is important that they should remain in alignment and withstand the constant dynamic loading to which they are subjected. Experience has shown that they are not free from defects, which require continual maintenance, and the

*Paper to be read before a Joint Meeting of the Institution of Structural Engineers, the British Iron and Steel Research Association and the Engineers' Group of the Iron and Steel Institute at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 22nd, 1953, at 6 p.m.

This Paper is prepared from Papers PE/E/44/52 and PE/E/45/52 of the Civil Engineering Committee of the Plant Engineering Division of the British Iron and Steel Research Association. Mr. Terrington is Head of the Civil Engineering Section of the Association and Dr. Hawkes is a Senior lecturer at the Royal Military College of Science, and formerly on the staff of B.I.S.R.A.

ere is scope for improvement in design. Following study of details of design in the light of present-day gulations, recent research work and modern experience, is paper, which has been prepared from two papers(2) &), published by The British Iron and Steel Research ssociation (B.I.S.R.A.), makes recommendations, where ossible, for the best design criteria for crane gantry rders, particulary the heavy types required in steel-orks. A considerable amount of factual information n existing girders in addition to longer discussions on ie various issues, are given in the B.I.S.R.A. papers to hich readers are referred.

Type and Weight of Girder

Spans of crane gantry girders in melting shops range om about 25 ft. to about 120 ft., and existing types clude lattice girders and plate girders with both one id two webs.

Objections to lattice girders include the large secondary resses developed at the intersections of the various

and transport limitations. These factors can be allowed for when the depth for a girder of least-weight is known and this is given by the following equation :—

$$D = 2.69^{*}\ 3\sqrt{W_E . L}\ \text{ins.}$$

where D = "least-weight" depth of girder in inches.

W_E = total equivalent uniformly distributed load in tons.

L = span in feet.

Values of the depth D have been calculated from this equation for various values of the equivalent uniformly distributed load (E.U.D.L.) and are shown plotted against the span in Fig. 1.

As the full loading conditions of the existing girders examined are not known, it is not practicable to show how their depths compare with the value given by the above equations.

N.B. The equivalent uniformly distributed load includes the deadweights of the girder.

**Some authorities prefer this factor to have the value of 3.0*

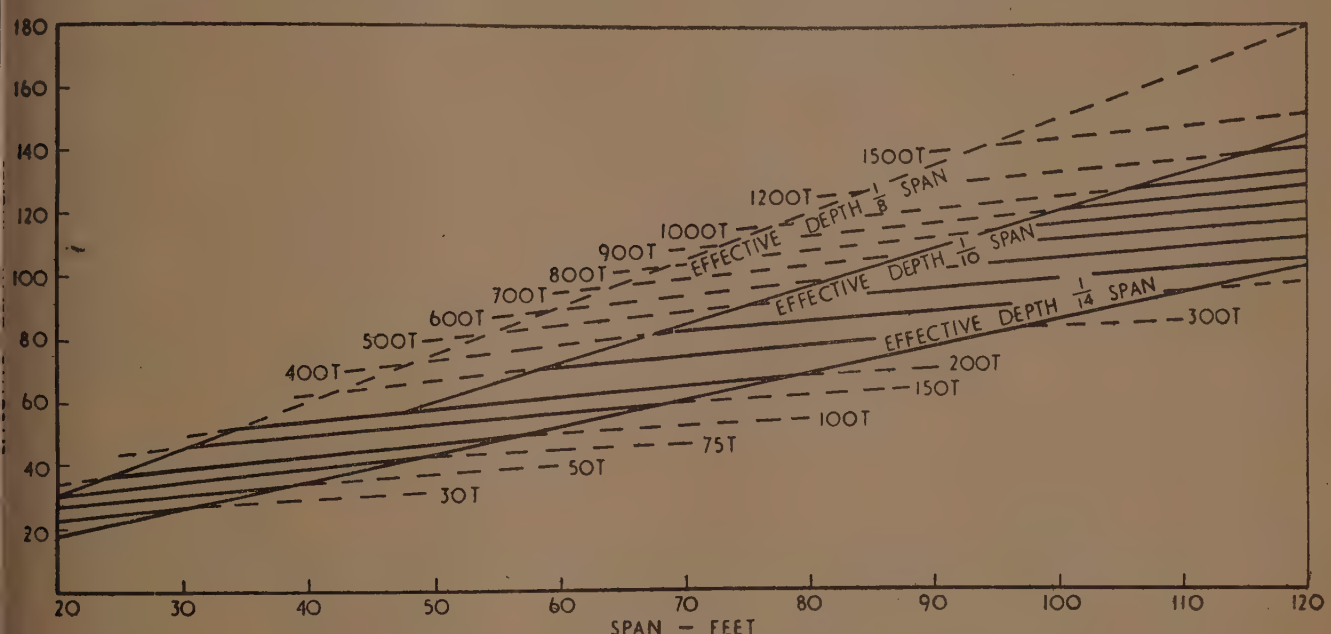


Fig. 1.—Effective depth of girder

embers ; the difficulty of providing a suitable section o resist the local bending stresses induced by the wheel oads in the top chord ; and the tendency of the wheel oad to shear the rail through the top plate of the girder.

Objections to plate girders with two webs are the endency of the wheel loads to shear through the top late of the girder ; the inaccessibility of the girder for naintenance inside the webs ; and the impossibility f renewing the rivets when they become loose. In rder to reduce weight, consideration might be given to tiffened suspended girders. Over long spans, however, irders such as these would scarcely be rigid enough, articularly in a lateral direction in which they have to esist considerable surge forces from the cranes.

With the improved methods of fabrication including velding, plate girders having single webs, are considered o be the most satisfactory and economical.

Recommendations by various authorities (6)(7)(8)(9) for irder depth range from about 1/14 to 1/8 of the span ; the atios of girders examined varied from 1/19.33 to /5.63, the majority being between 1/12 and 1/8. The ptimum depth depends on many factors, including ntensity and type of loading, the available headroom, he cost of fabrication, differential deflection between irders on opposite sides of the building, shop equipment

Flanges

Introduction

The flanges have to carry their own direct stresses in tension and compression and, in addition, they have to stiffen the web against lateral buckling. The tension flanges do not present any difficulty but the compression flanges have a tendency to buckle under load. This is dependent on the intensity of stress to which they are subjected and to the slenderness of their cross-section in relation to their unsupported length. For any given girder section, therefore, the object is to use as high a stress as possible but sufficiently low to avoid actual collapse due to buckling, whilst allowing a suitable factor of safety.

General Principles

Principal considerations in the design of compression flanges are :—

(i) They should not be too wide because the bending stress is not constant(8) but diminishes towards the edges as shown in Fig. 2. Material at the edges of a very thin wide flange only therefore carries a small bending stress. If the design assumes a constant stress distribution (shown by the dotted line), the actual maximum stress will be much in excess of the design stress, and in

addition, material at the outer edges is not being used economically.

(ii) They must not be too thin in relation to their width, otherwise the flanges themselves will buckle. To avoid buckling, it is generally accepted [9] [10] [11] [12]

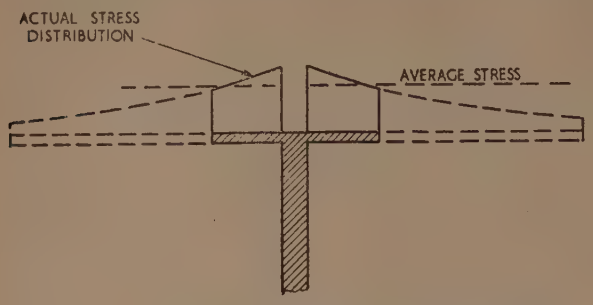


Fig. 2.—Stress Distribution across flange

that the projection of the plate beyond the point at which the flange is secured, should not be excessive. The extent to which the flange may project is dealt with in subsequent sections.

(iii) They must not be too narrow otherwise there will be excessive lateral deflection due to lack of lateral rigidity. A lower limit recommended for use in practice is 1/30 × span and the corresponding calculated value has been evaluated as 1/40 × span[13].

The ultimate selection of a section is inevitably a compromise between these two opposing tendencies.

PERMISSIBLE MAXIMUM FIBRE COMPRESSIVE STRESS

There are three ways in which a flange may fail, giving rise to the following considerations :—

(i) The elastic stability of the girder as a whole, which comprises the torsional resistance of the section together with the resistance of the flanges to lateral bending.

(ii) The stability of the flange to avoid wrinkling of an unstiffened edge between stiffeners.

(iii) The stability of the compression flange as a whole, to avoid angular distortion of the compression flange between stiffeners.

Considering these effects separately :—

(i) THE ELASTIC STABILITY OF THE GIRDER AS A WHOLE

The section may deflect and distort completely, as in Fig. 3 and analysis[3], [16] shows that the permissible stress f_{b1}, under these conditions, is expressed by :—

$$f_{b1} = \left(\frac{\pi}{4} \cdot \frac{1}{I_{xx}} \cdot \frac{d_1}{L}\sqrt{EI_{yy}GJ}\right)\left(\sqrt{1 + \frac{\pi^2}{4} \cdot \frac{EI_{yy}}{GJ}\left(\frac{d_1}{L}\right)^2}\right) \quad \ldots\ldots (5)$$

Fig. 3—ELASTIC DEFLECTION

Fig. 3

In this equation, the portion in the first bracket expresses the torsional resistance of the girder as a whole and the portion in the second bracket expresses the increase in the allowable stress due to warping of the section during torsion, namely, the resistance the flanges in lateral bending.

Now equation (5) has already been made use of in t simplified semi-empirical equation given in Clause 1 B.S.449, 1948, but a new development of this ba equation (5) has been put forward [3] by one of t authors, which permits closer adherence to theory, wi advantages in some instances. It will be appreciat that comparison is made with B.S.449 because B.S.4 is the latest and best regulation hitherto available.

First regard ng B.S.449, 1948, Clause 19c[4], [5], it will seen that the equation given there employs only t portion in the first bracket thus :—

$$f_{b1} = \frac{\pi}{4} \cdot \frac{d_1}{I_{xx} \cdot L}\sqrt{E \cdot I_y \cdot G \cdot J} \quad \ldots\ldots (1$$

$$= 1{,}000 \frac{k_{yy}}{L} \cdot K_1 \quad \ldots\ldots (1$$

where $E = 13{,}400$

Poisson's ratio, $\mu = 0.3$

$$\frac{d_1 A}{k_{xx}^2 \; k_{yy}} = 4.65$$

$$\frac{k_{xx}}{k_{yy}} = 4.9$$

where $\frac{k_{xx}}{k_{yy}} = 5.0 \rightarrow 3.0$

& $K_1 = 1.0 \rightarrow 1.5$

K_1 is an allowance for the stockier or heavier bea sections.

The alternative suggested equation[3] makes use of t complete basic equation (5) thus :—

$$f_{b1} = \left(\frac{\pi}{4} \cdot \frac{1}{I_{xx}} \cdot \frac{d_1}{L}\sqrt{E \cdot I_{yy} \, G \cdot J}\right)\left(\sqrt{1 + \frac{\pi^2}{4} \cdot \frac{E.I_{yy}}{G.J}\left(\frac{d_1}{L}\right)^2}\right) \quad \ldots\ldots (1$$

$$= \frac{1}{4\sqrt{10}} \cdot E \cdot F_2^2 \sqrt{\frac{1}{F_1}} \cdot \frac{d_1}{k_{yy}}\sqrt{1 + \frac{5\pi^4}{2} F_1 \left(\frac{d_1}{L}\right)^2} \cdot \frac{k_{yy}}{L} \quad \ldots (1$$

where $F_1 = \dfrac{k_{yy}^2 \; (k_{xx}^2 + k_{yy}^2)}{A^2}$

$$F_2 = \frac{k_{yy}}{k_{xx}}$$

Poisson's ratio

$\mu = 0.3$

$G = 0.4 \; E.$

$$J = \frac{1}{4\pi^2} \cdot \frac{A^4}{(I_{xx} + I_{yy})}$$

NOTE : Where the section is asymmetrical and t distance from the outer fibre of the compression flan to the centre of area is O.X. of the depth of the girde

then multiply f_{b1} by $\dfrac{O.X}{0.5}$

Comparison of equations (7) and (8) shows that the lue of K_1 in the B.S.449 equation (7) has an analogous lue in the newly developed equation (8) and is :—

$$K_1 = \frac{1}{1{,}000} \cdot \frac{1}{4\sqrt{10}} \cdot E.F_2^2 \Big/ \sqrt{\frac{1}{F_1}} \cdot \frac{d_1}{k_{yy}} \sqrt{1 + \frac{5}{2}\pi^4 F_1 \left(\frac{d_1}{L}\right)^2}$$

in Clause 19c, B.S.449, 1948, are shown as horizontal lines for the various values of β from 3.0 to 5.0. It will be seen that, whereas the B.S.499 equation suggests that K_1 varies only with $\frac{k_{xx}}{k_{yy}}$ (β), K_1 actually varies with the values of $\frac{F_1}{F_2}$ and the values of $\frac{L}{k_{yy}}$.

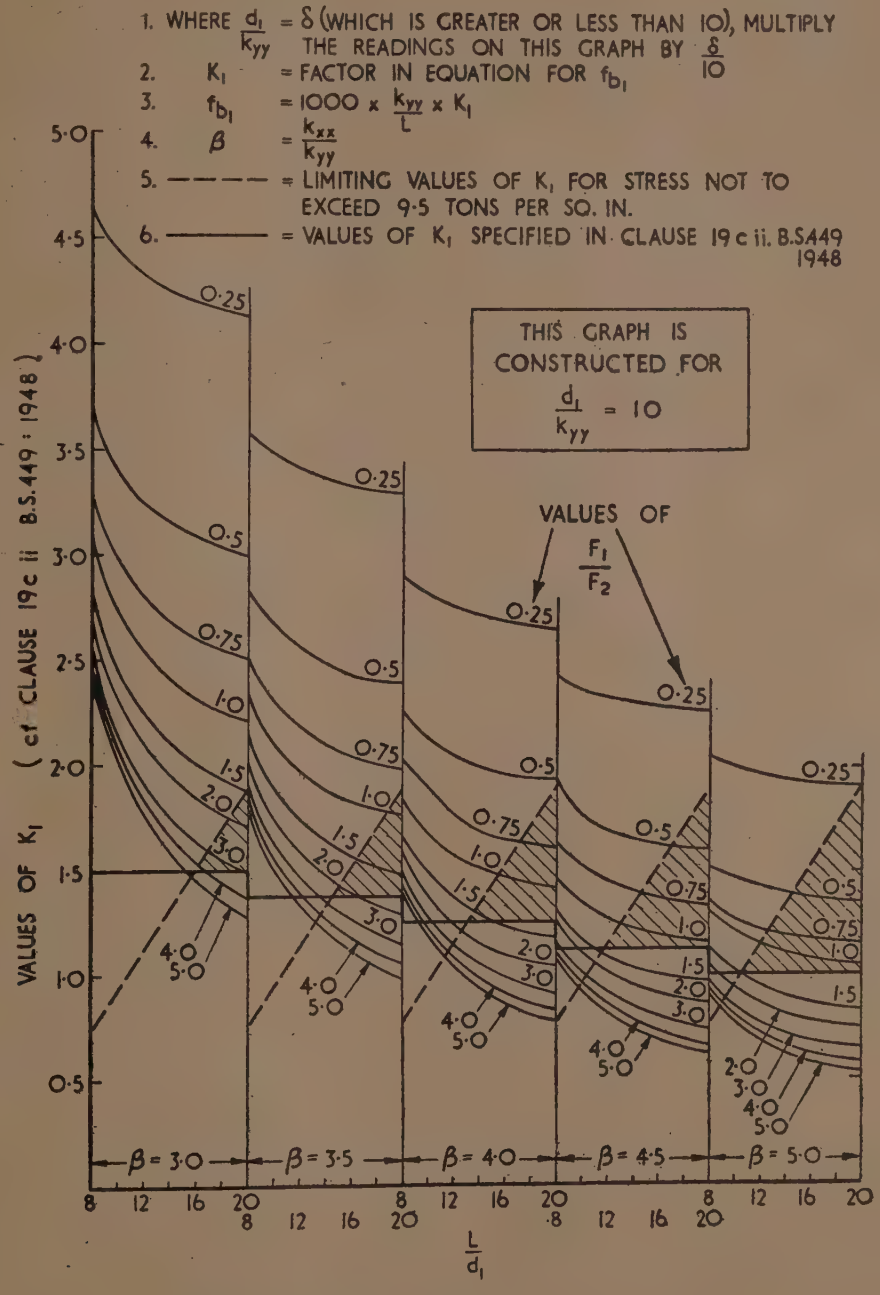

1. WHERE $\frac{d_1}{k_{yy}}$ = δ (WHICH IS GREATER OR LESS THAN 10), MULTIPLY THE READINGS ON THIS GRAPH BY $\frac{\delta}{10}$
2. K_1 = FACTOR IN EQUATION FOR f_{b_1}
3. f_{b_1} = 1000 x $\frac{k_{yy}}{L}$ x K_1
4. β = $\frac{k_{xx}}{k_{yy}}$
5. ———— = LIMITING VALUES OF K_1 FOR STRESS NOT TO EXCEED 9·5 TONS PER SQ. IN.
6. ———— = VALUES OF K_1 SPECIFIED IN CLAUSE 19 c ii. B.S.449 1948
THIS GRAPH IS CONSTRUCTED FOR $\frac{d_1}{k_{yy}}$ = 10
VALUES OF $\frac{F_1}{F_2}$
VALUES OF K_1 (cf CLAUSE 19c ii B.S.449 : 1948)
β = 3·0
β = 3·5
β = 4·0
β = 4·5
β = 5·0
$\frac{L}{d_1}$

Fig. 4.—Quantity K_1 compared with $\frac{L}{d_1}$

This expression for K_1 is shown in graph form in ig. 4, for values of $\frac{d_1}{L}$ from 8 to 20, where $\frac{d_1}{k_{yy}}$ = 10 n average value for B.S. beams) for ranges of = $\frac{k_{xx}}{k_{yy}}$ from 3.0 to 5.0 and for ranges of $\frac{F_1}{F_2}$ from .25 to 5.0. The corresponding values of K_1 as specified

The diagonal lines in the graph (Fig. 4) show the limiting values of K_1 corresponding to a maximum normal fibre stress of 9.5 tons per square inch, which is the maximum stress permitted in plate girders by B.S.449. Suitable adjustment of these lines can be made for high tensile steels and steels having guaranteed yield stresses other than that of mild steel as specified in B.S.449. Depending on the value of $\frac{F_1}{F_2}$ between

these diagonal and horizontal lines, the value of K_1, and hence the maximum permissible fibre stress, can be increased in excess of the value permitted by B.S.449. For example, the possible increase, depending on the values of $\frac{F_1}{F_2}$ and $\frac{L}{k_{yy}}$, can range up to 90 per cent., where $\frac{k_{xx}}{k_{yy}} = 5.0$ and up to 13 per cent. approx. where $\frac{k_{xx}}{k_{yy}} = 3.0$. The increased permissible stress in the stockier sections is illustrated in Fig. 5.

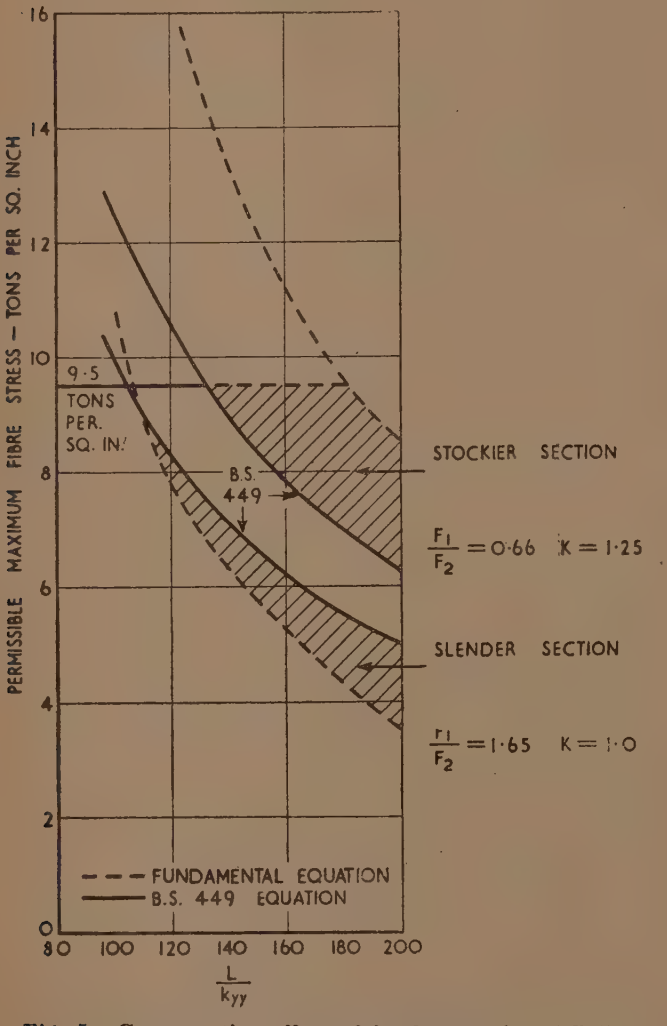


Fig. 5.—Comparative effect of fundamental and B.S. 449 equations on sections of differing robustness

To use Fig. 4 for any given example, calculate the value of $\frac{d_1}{k_{yy}} = \delta$ (say) then :—

the true value of K_1 = (value from graph in Fig. 4) $\times \frac{\delta}{10}$

PERMISSIBLE FLANGE FIBRE STRESS f_{b_1} (Fig. 6)

Fig. 6 has been prepared to enable the actual permissible flange fibre stress f_{b_1} to be read off the graph for given values of L/k_{yy} and K_1. The value of K_1 may either be the value specified by B.S.449 or the true value of K_1 as previously ascertained from Fig. 4 (and corrected for d_1/k_{yy} as necessary).

(ii) STABILITY OF THE COMPRESSION FLANGE BETWEEN STIFFENERS

Where a flange is unstiffened at the edges, it must be prevented from wrinkling between the web stiffeners, by limiting the permissible stress in relation to the unsupported flange projection as in Fig. 7 below.

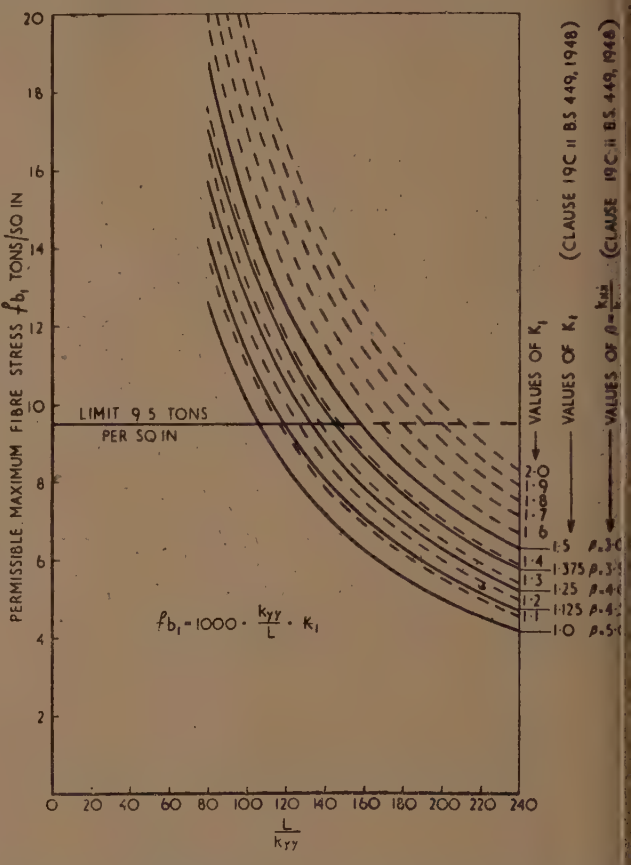

PERMISSIBLE MAXIMUM FIBRE STRESS f_{b_1} TONS/SQ IN
LIMIT 9·5 TONS PER SQ IN
VALUES OF K_1
(CLAUSE 19C II B.S. 449, 1948)
$f_{b_1} = 1000 \cdot \frac{k_{yy}}{L} \cdot K_1$
$\frac{L}{k_{yy}}$

Fig. 6.—Permissible maximum fibre stress f_{b_1}— elastic stability of the complete girder

Fig. 7. FLANGE STIFFNESS

Analysis has shown ([17]) this to be :—

$$f_{b_2} = \frac{1.238}{2} \cdot \frac{\pi^2 E}{12} \left(\frac{t_1}{b_1}\right)^2 \quad \ldots \ldots \quad (9$$

For reference, values of f_{b_2} are plotted in Fig. 8.

Corresponding approximate formulæ with the similar object of avoiding local flange buckling are :—

Regulation	b_1 not less than	Edge Condition
B.S. 449 : 1948	$16t_1$	Unstiffened
,,	$20t_1$	Stiffened
American Association of Iron and Steel Engineers Specification for Electric Overhead Travelling Cranes.	6 in. or $12T\sqrt{\frac{.17000}{fc}}$	do.

T = plate thickness in inches.
fc = maximum (actual) compression stress in lb. per sq. in.

(iii) Stability of the Compression Flange to avoid Angular Distortion between Stiffeners

The compression flange under stress may distort nolithically with the compressed portion of the web an angle to their normal position, as in Fig. 9.

g. 9—ANGULAR DISTORTION OF COMPRESSION FLANGE

Under these conditions, the limiting stress is expressed [17] :—

$$f_{b3} = \tfrac{1}{2}\left[\frac{E}{1+6.28\,.\,10^4\,.\,B^3\,.\,C^4}\left(1.6B^2+B^3(104C^4-1.008)+\frac{2B^2C}{19.75\sqrt{}}\sqrt{\frac{1}{B}+2.51\,.\,10^5B^2C^6}\right)\right] \quad . \quad . \quad . \quad . \quad . \quad . \quad (10)$$

For reference, values of f_{b3} are plotted in Fig. 10.

$$B = \frac{t_1}{b} \quad C = \frac{t_2}{t_1}$$

On the principle that the component units must be at ast as strong as, and in practice stronger than, the

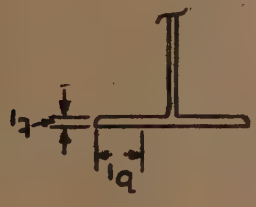


Fig. 7

mplete structure, the values of f_{b2} and f_{b3} must ways be greater than f_{b1}. For a complete assessment the permissible stress, equations (8), (9) and (10) ust therefore be used simultaneously.

pplication to Design

To make use of the foregoing theory the various stages ould be :—

(1) The span is usually given or determined.

(2) The depth for least-weight is calculated as escribed on page 269.

(3) The required section modulus is calculated from e loading, using a maximum fibre stress of 9.5 tons er sq. in.

(4) From this point three courses can be taken :—

Method 1. Select a section with the required modulus section and from equation (8) already described, lculate the permissible stress in bending. Alternatively, use the graphs in Figs. 4 and 6. If the section not satisfactory, select other sections in turn until one found to be suitable.

Method 11. Without actually calculating the permissible stress, have some method of testing whether e section selected will withstand the maximum mpressive stress, namely 9.5 tons per sq. in. This as been made possible by means of the graph in Fig. 11.

Method 111. Without actually calculating the permissible stress, have some method of selecting the section directly with the knowledge that it will withstand the maximum compressive stress. A suggestion for this is illustrated in Fig. 12.

In all cases the section selected must be checked by equations (9) and (10) to ensure that local buckling will not occur.

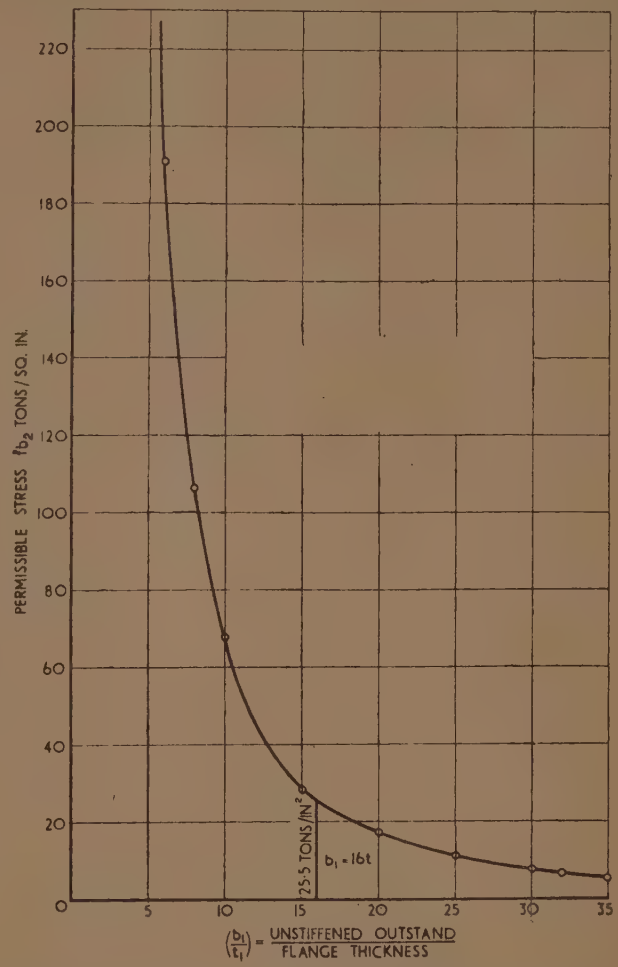


Fig. 8.—Permissible maximum fibre stress f_{b_2} to avoid wrinkling of an unstiffened edge between stiffeners

Turning to the three methods :—

Method 1. The use of the new equation (8) or Figs. 4 and 6 will be sufficiently clear without further elaboration.

Method 11. From equation (8), it is seen that the permissible stress depends not only on the properties of the cross-section but also the ratio of the depth-to-length between effective lateral restraints, and that it is made more complex by the last portion, $\sqrt{1+\frac{5}{2}\pi^4\,.\,F_1\left(\frac{d_1}{L}\right)^2}$ namely, the increase in permissible stress due to the lateral resistance of the flanges in bending. However, it has been shown in Fig. 11 that the efficiency η, or the "shape factor" per unit depth where, $\eta = \frac{s_t}{d_1} = \frac{1}{2\sqrt{10}}F_2^2\sqrt{\frac{I}{F_1}}$, of any section may still be used as a basis of comparing sections and testing whether they can withstand the permissible stress of 9.5 tons per sq. in. The method of constructing the graph is shown in the figure, namely by choosing values

of $f_{b1} = 9.5$ tons per sq. in., and $\frac{d_1}{L}$, and then plotting the value which the "shape factor" per unit depth $\frac{s_t}{d_1}$ should

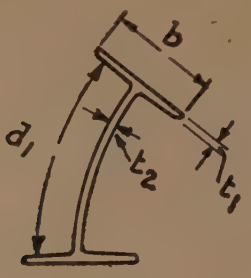


Fig. 9

have for these given values of f_{b1} and $\frac{d_1}{L}$. Hence for any section, knowing the values of its $\frac{s_t}{d_1}$ and F_1 if these are plotted on Fig. 11, they should occur on the known $\frac{d_1}{L}$ curve if they are to withstand 9.5 tons per sq. in. To facilitate this, it is suggested that in future all steel handbooks should have the values of F_1, F_2 and $\frac{1}{2\sqrt{10}}$

$F_2^2 \sqrt{\frac{I}{F_1}}$ listed against each section.

Method III. As one point on the graph in Fig. 11 will represent any number of sections, it is possible to give particular solutions if the number of variables is limite As an example, sections of one type and one depth a given in Fig. 12, in which a whole range of sections h been plotted, showing contour lines indicating :—

(a) flanges of equal width and varying thickness.
(b) flanges of equal thickness and varying width.
(c) flanges of varying width and thickness, all of which separately provide the same modulus of section.

The graph also shows the increase or decrease in t size of flange, to keep the maximum fibre stress consta at 9.5 tons per square inch, as the span increases decreases.

For practical design, it is suggested that a series these sheets should be prepared. Each sheet wou relate to a range of sections of a particular type a stated depth and all would be valid for a limiting stre of 9.5 tons per sq. in. The graphs would be segregat into types and the depths would vary by about 6 in. (say Perusal of the graphs would facilitate the selection of number of possible sections which would provide t required modulus of section on a given length betwe effective lateral restraints.

The series of graphs prepared once and for all woul it is suggested, be bound together in the form of a boo for ready reference. Any slight variations necessa from any of the standard solutions could be mad rapidly by slide-rule, but the sections given in t graphs would provide a close, if not indeed, an exac answer. Any initial extra trouble in a design office

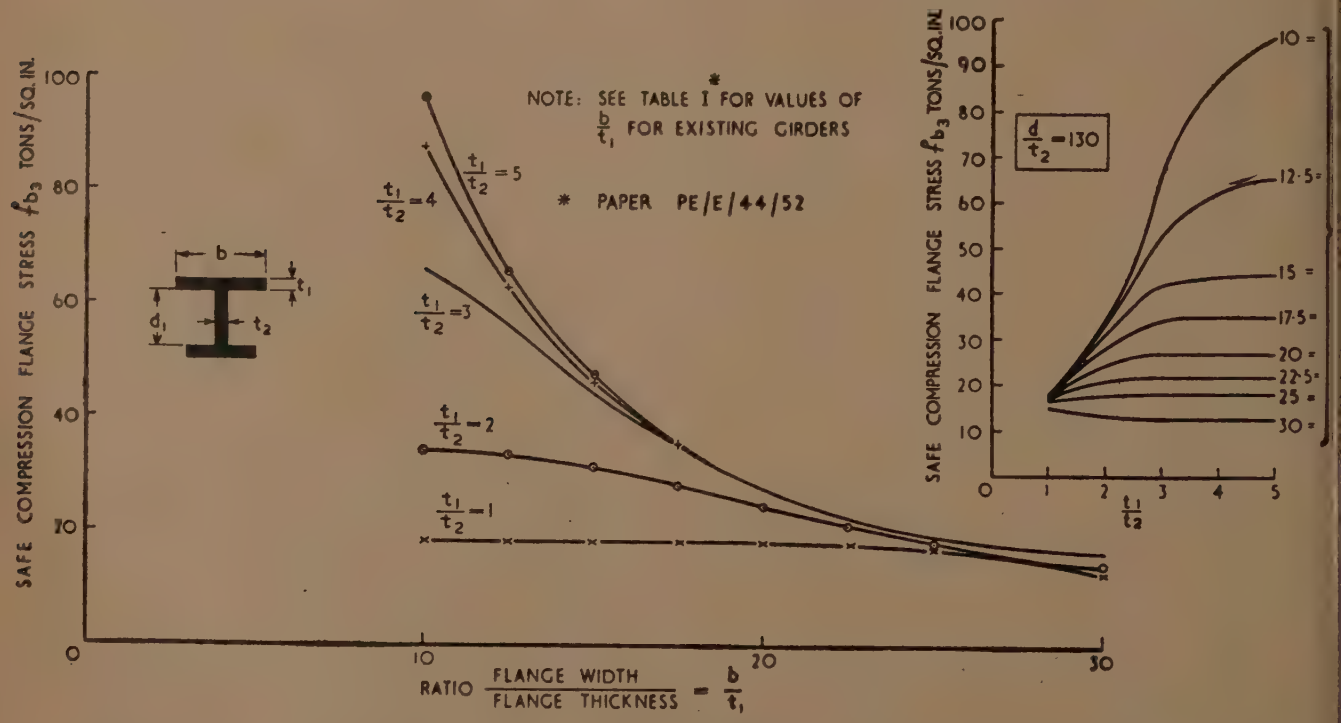


Fig. 10.—Permissible maximum fibre stress f_{b_3} to avoid angular distortion of the compression flange between stiffeners

the preparation of the graphs would be repaid by th graphs being available for constant reference. At th time of writing this paper, it has not been practicab to prepare a number of these graphs to illustrate mo clearly the advantages to be gained.

Analysis of Existing Girders

Analysis of existing girders(2) shows that the propo tions of their cross-section have a wider range of slende ness than those of British Standard Beams, and that th

permissible stress for gantry girders as given by B.S.449 can differ radically from the values by the fundamental equation ; that is the value of K_1 as given by B.S.449 can be too low for the stockier sections and too high for the more slender sections as compared with the fundamental equation. Moreover, the value of K_1 also varies with the area of cross-section and the ratio of lateral slenderness L/k_{yy} which is not sufficiently emphasised in B.S.449.

Possible Sections

The various types of sections which should be calculated for use on the graphs described above, are shown in Fig. 13.

The desirability of concentrating metal over the web has been met in riveted work by making the area of the flange angles as large as possible. This is catered for by the general rule that the flange angle area should be at least one-third of the total flange area as shown in Fig. 13 (i) to (iv). Another method employed in riveted work is the use of double web plates, Fig. 13 (ii) to (iv). In welded girders the joint is in some cases splayed between the web and the flanges [18], Fig. 13 (v) to (vi), or a flitch plate used as in Fig. 13 (vii).

The desirability of dispersing the metal over a wide flange has been met by the use of flat plates Fig. 13 (viii), bulb angles (ix), edge angles (x), channels (xi), joists (xii), twin channels (xiv) and welded flats as edge stiffeners (xv).

In order to assess which is the most economical type of section, all the above types would require to be calculated and plotted graphically as in Fig. 12. The cross-

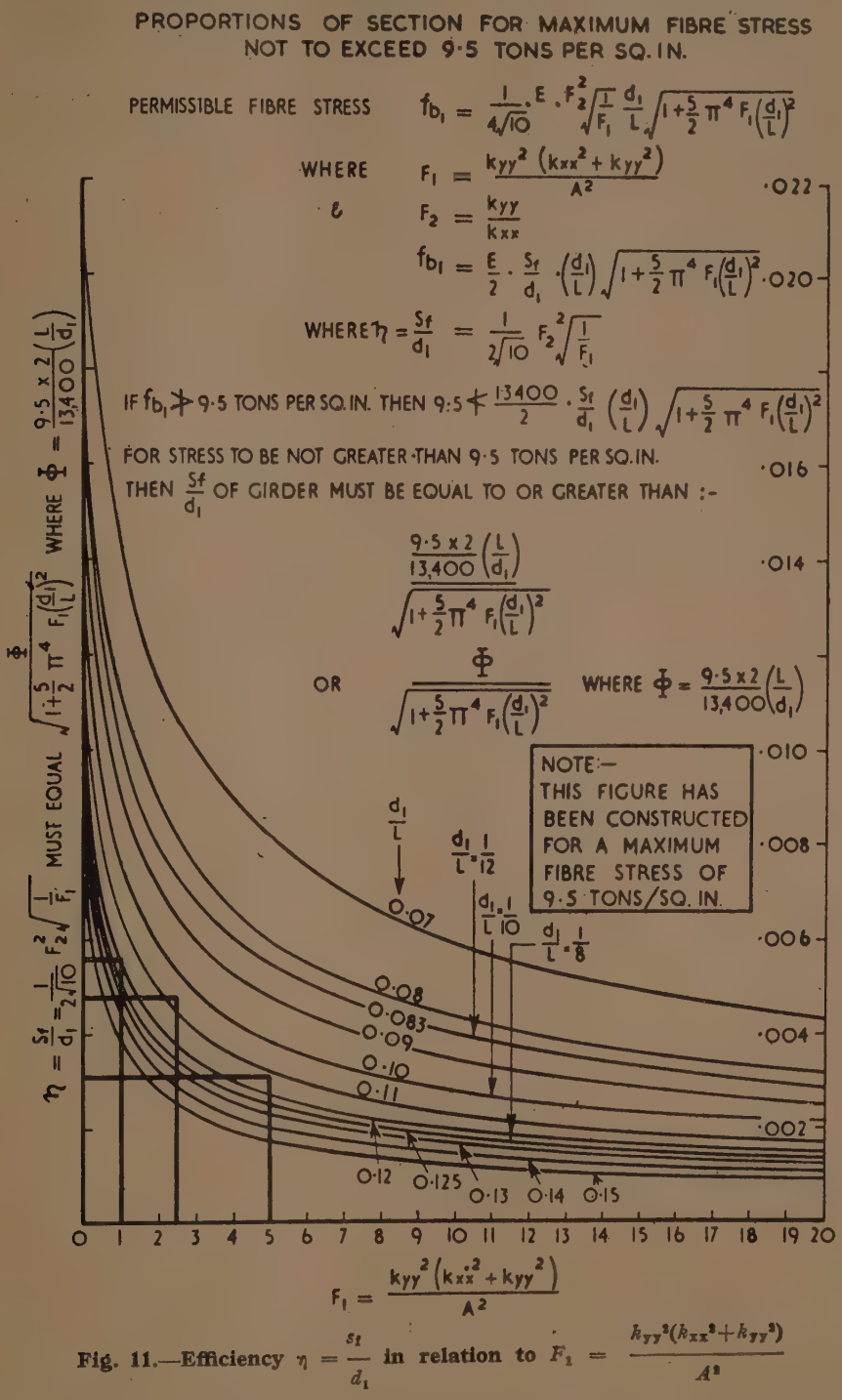

PROPORTIONS OF SECTION FOR MAXIMUM FIBRE STRESS NOT TO EXCEED 9·5 TONS PER SQ. IN.
PERMISSIBLE FIBRE STRESS $f_{b_1} = \frac{1}{4\sqrt{10}} \cdot E \cdot F_2^2 \sqrt{\frac{1}{F_1}} \frac{d_1}{L} \sqrt{1+\frac{5}{2}\pi^4 F_1 \left(\frac{d_1}{L}\right)^2}$
WHERE $F_1 = \frac{k_{yy}^2 (k_{xx}^2 + k_{yy}^2)}{A^2}$
& $F_2 = \frac{k_{yy}}{k_{xx}}$
$f_{b_1} = \frac{E}{2} \cdot \frac{S_f}{d_1} \cdot \left(\frac{d_1}{L}\right) \sqrt{1+\frac{5}{2}\pi^4 F_1 \left(\frac{d_1}{L}\right)^2}$
WHERE $\eta = \frac{S_f}{d_1} = \frac{1}{2\sqrt{10}} F_2^2 \sqrt{\frac{1}{F_1}}$
IF $f_{b_1} \not> 9{\cdot}5$ TONS PER SQ. IN. THEN $9{\cdot}5 \not< \frac{13400}{2} \cdot \frac{S_f}{d_1} \left(\frac{d_1}{L}\right) \sqrt{1+\frac{5}{2}\pi^4 F_1 \left(\frac{d_1}{L}\right)^2}$
FOR STRESS TO BE NOT GREATER THAN 9·5 TONS PER SQ. IN.
THEN $\frac{S_f}{d_1}$ OF GIRDER MUST BE EQUAL TO OR GREATER THAN :-
$\frac{\frac{9{\cdot}5 \times 2}{13{,}400}\left(\frac{L}{d_1}\right)}{\sqrt{1+\frac{5}{2}\pi^4 F_1 \left(\frac{d_1}{L}\right)^2}}$
OR $\frac{\Phi}{\sqrt{1+\frac{5}{2}\pi^4 F_1 \left(\frac{d_1}{L}\right)^2}}$ WHERE $\Phi = \frac{9{\cdot}5 \times 2}{13{,}400}\left(\frac{L}{d_1}\right)$
NOTE— THIS FIGURE HAS BEEN CONSTRUCTED FOR A MAXIMUM FIBRE STRESS OF 9·5 TONS/SQ. IN.
$\eta = \frac{S_f}{d_1} = \frac{1}{2\sqrt{10}} F_2^2 \sqrt{\frac{1}{F_1}}$ MUST EQUAL $\frac{\Phi}{\sqrt{1+\frac{5}{2}\pi^4 F_1 \left(\frac{d_1}{L}\right)^2}}$ WHERE $\Phi = \frac{9{\cdot}5 \times 2}{13{,}400}\left(\frac{L}{d_1}\right)$
$\frac{d_1}{L}$
$\frac{d_1}{L} = \frac{1}{12}$
$\frac{d_1}{L} = \frac{1}{10}$
$\frac{d_1}{L} = \frac{1}{8}$
0·07
0·08
0·083
0·09
0·10
0·11
0·12
0·125
0·13
0·14
0·15
·022
·020
·016
·014
·010
·008
·006
·004
·002
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20
$F_1 = \frac{k_{yy}^2 (k_{xx}^2 + k_{yy}^2)}{A^2}$

Fig. 11.—Efficiency $\eta = \frac{s_f}{d_1}$ in relation to $F_1 = \frac{k_{yy}^2(k_{xx}^2+k_{yy}^2)}{A^2}$

sectional area provided and the cost of fabricating the section should then be plotted against the modulus of section for a given span, and this should give an idea of the relative cost in money and material. At the time of preparing this paper, it has not been practicable to pursue this.

Alternatively, the efficiency η or shape factor per unit depth, should be plotted against the cross-sectional area.

Flange Splices

Splices would preferably be placed at a point of least stress which in practice is near the end of the span since the flange stress is not uniformly the maximum permissible at all points in the span. As a compromise, the following arrangements are considered the best :—

Up to 40 ft. span : flanges in one length.

From 40 ft. to 80 ft. span : position of joint will be from 1/4th to 1/5th of the span from the end.

Above 80 ft. span : position of joint will be governed by the maximum available length of plate, viz. :

Up to 40 ft. (for thick wide plates (rolling limit).

Up to 60 ft. (for thin plates and for medium plates) (handling limit)

Fig. 14 shows the position of the joints in a number of existing girders. It will be seen that in the majority of cases the joints are near the centre of the girder and in some cases actually at the centre. This construction is considered to be unsatisfactory.

A fabrication difficulty which should be borne in mind is that joint cover plates used in the top flange of riveted girders will often necessitate additional packing plates to maintain a level surface for the crane rail. The best solution to this problem would appear to be to splice the flange in conjunction with plate curtailment, using an additional length of top plate as a cover plate for the lower flange plate. This difficulty does not of course arise in welded girders, where the flanges are butt jointed.

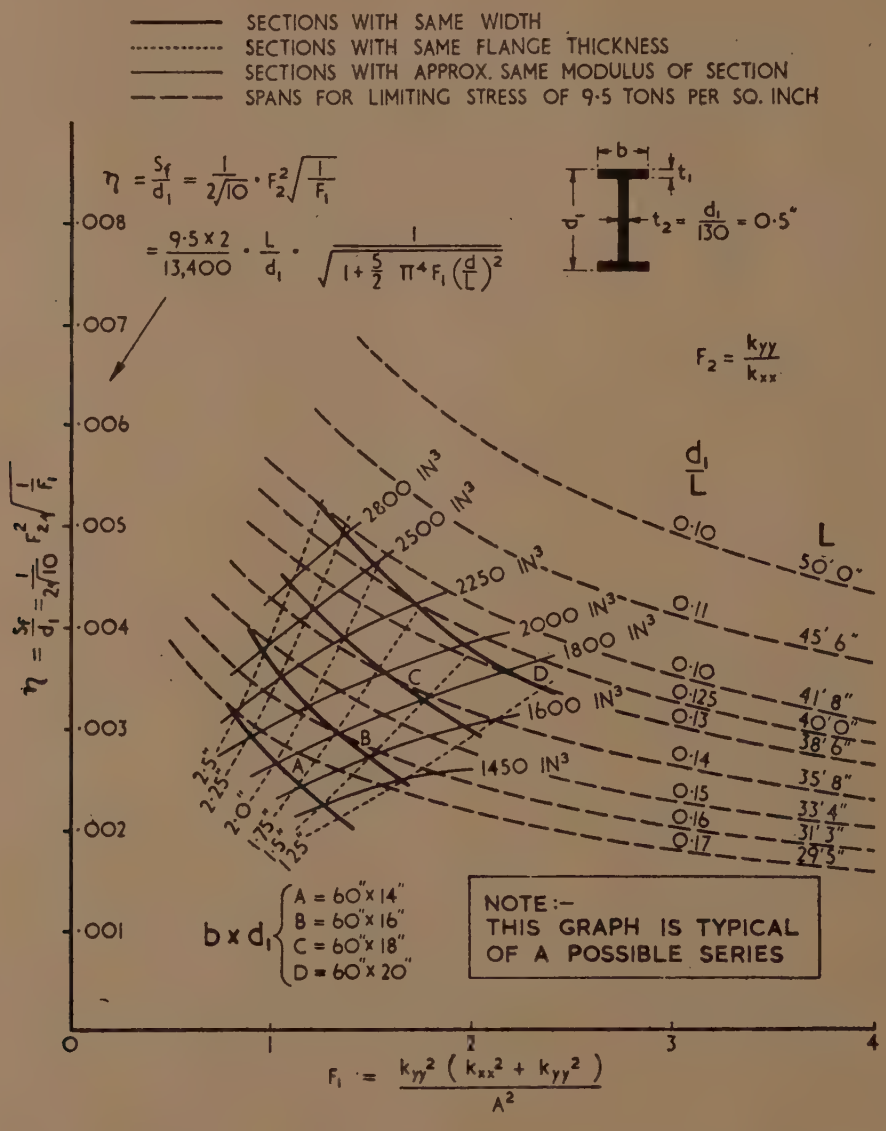

SECTIONS WITH SAME WIDTH
SECTIONS WITH SAME FLANGE THICKNESS
SECTIONS WITH APPROX. SAME MODULUS OF SECTION
SPANS FOR LIMITING STRESS OF 9·5 TONS PER SQ. INCH
$\eta = \frac{S_f}{d_1} = \frac{1}{2\sqrt{10}} \cdot F_2^2 \sqrt{\frac{I}{F_1}}$
$= \frac{9{\cdot}5 \times 2}{13{,}400} \cdot \frac{L}{d_1} \cdot \frac{1}{\sqrt{1+\frac{5}{2}\pi^4 F_1 \left(\frac{d}{L}\right)^2}}$
$t_2 = \frac{d_1}{130} = 0{\cdot}5"$
$F_2 = \frac{k_{yy}}{k_{xx}}$
$F_1 = \frac{k_{yy}^2 (k_{xx}^2 + k_{yy}^2)}{A^2}$
2800 IN³
2500 IN³
2250 IN³
2000 IN³
1800 IN³
1600 IN³
1450 IN³
A = 60"x14"
B = 60"x16"
C = 60"x18"
D = 60"x20"
NOTE :— THIS GRAPH IS TYPICAL OF A POSSIBLE SERIES

Fig. 12.—Typical graph of range of sections 60 inches deep (one of a series)

Webs

Introduction

The stresses carried by the web are largely due to shear, so that in earlier years their thickness was usually arrived at by assuming a value for the working shear stress. Considerations concerning their stability, however, provide a more fundamental basis for design. Most modern specifications have considered this aspect

nd have prescribed limits to the thickness at some raction of the web depth :—

American Railway Association Specification	1931 web not less than 1/20 D (D = distance between flanges).

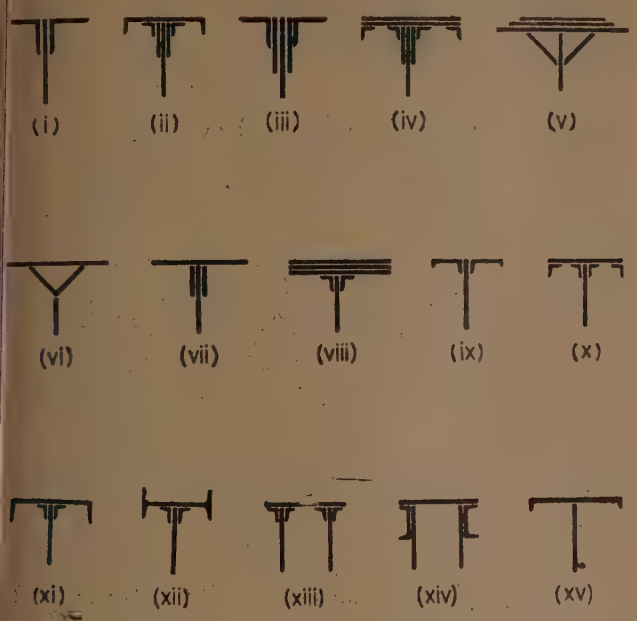


Fig. 13.—Types of compression flange

American Railway Association Specification	1934 web	not less than	1/170 web depth
American Institute of Steel Construction Specification	1946 web		
British Standard Specification 449	1948 web		

but in this paper, it is shown that the present limit should be altered.

A web plate is mostly stressed in tension across one diagonal and compression across the other. If it is very thin relative to its panel depth and breadth, diagonal buckling occurs at a very low load. When buckling occurs, the resistance of the plate to compression across the wave is slight and the load is largely carried by tensile stresses along the waves together with the reaction set up at the edges of the plate. Provided the tensile stresses are below the yield point of the material, the panel remains elastic and returns to its original form on removal of the load. If the yield point is exceeded, the panel continues to support a load almost as great as the buckling load, but will remain distorted after the load has been removed. If the thickness of the web is increased relative to the panel dimensions, the load, necessary to cause buckling, increases also until a point is reached where yielding of the material takes place before buckling occurs. Buckling, accordingly, does not necessarily amount to failure but is undesirable, and in conjunction with the elastic limit provides a basis for design. That is, the best web dimensions are those in which the panel will reach the yield point at the same load at which buckling commences.

Design Formulae

To determine a suitable web thickness, four limiting conditions of web loading have been considered and equations have been derived[2] for the resulting stress. The conditions are pure shear, pure bending, combined bending and shear and stresses under a point load, as follows :—

(1) Pure shear. This will occur at the extreme end of the girder,

$$\tau_{cr} = 12{,}105 \left(5{\cdot}35 + 4\left(\frac{s}{d}\right)^2 \eta_s\right) \frac{t^2}{s^2} \text{ tons/sq. in.}[22] \quad \ldots\ldots (1)$$

where

τ_{cr} = the critical shear stress at which buckling will occur.

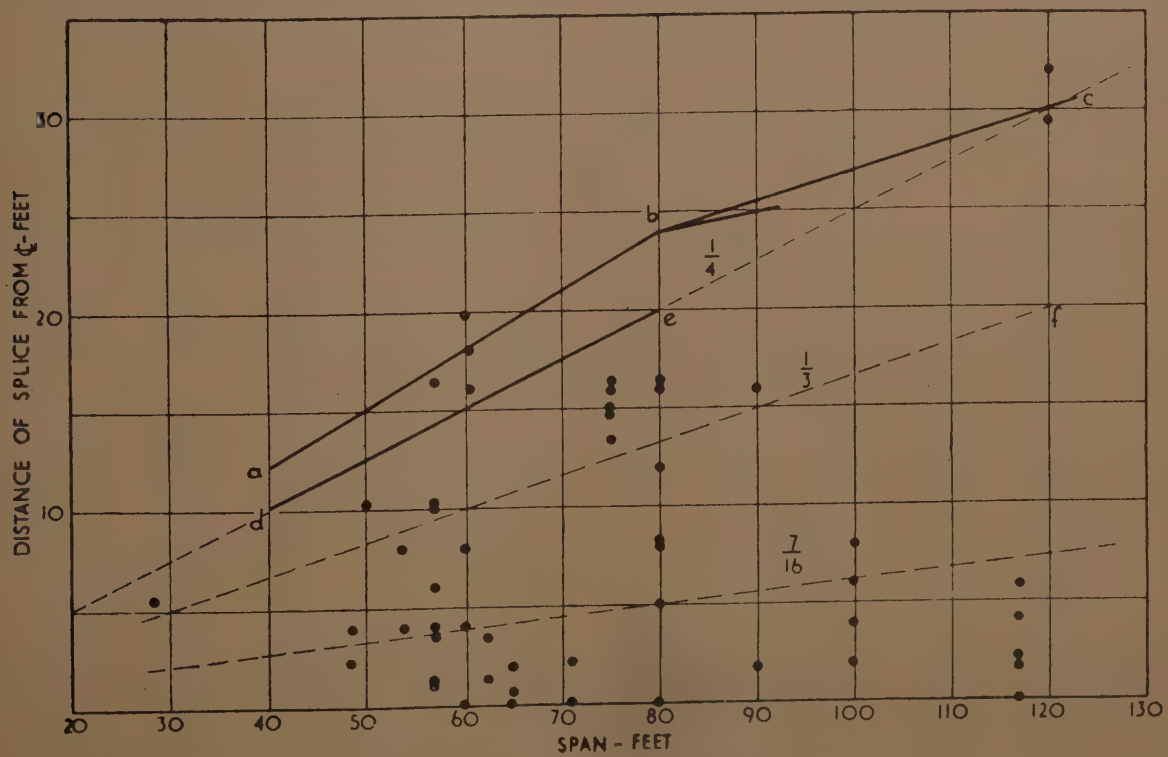


Fig. 14.—Position of flange splices

s = the stiffener spacing in inches.
d = unsupported depth of web in inches.
η_s = modulus factor in shear as given in Fig. 15.
t = web thickness in inches.
and the stiffener spacing (s) is not larger than the depth of the web (d).

(2) Pure Bending. This will occur under certain conditions of load over the central portion of the girder.

$$f_{cr} = 2.892997 \times 10^5 \, \eta_c \, \frac{t^2}{d^2} \text{ tons/sq. in. . . (2)}$$

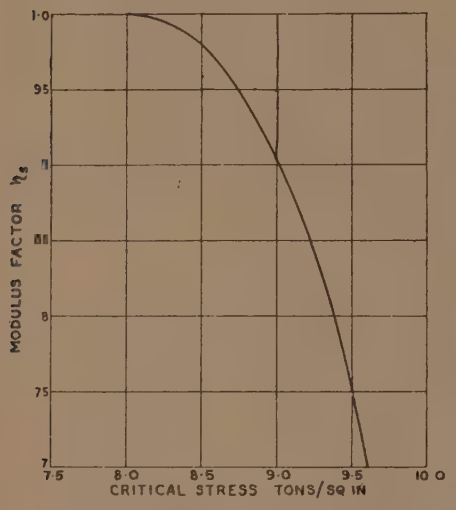


Fig. 15.—Modulus factor for shear, structural steel

where f_{cr} = the critical maximum compressive bending stress in the web at which buckling will occur.

η_c = modulus factor in compression as given in Fig. 16 [23].

(3) Combined bending and shear. Pure bending only occurs under certain conditions of load, the more general case being that of a web subjected to both bending and shear. In this case the design formula becomes

$$f_{cr} = 12{,}105 \, k \eta_c \, \frac{t^2}{d^2} \text{ tons/sq. in. . . (3)}$$

where f_{cr} = the critical maximum compressive bending stress at which buckling will occur.

k = a factor depending upon the ratio of stiffener spacing to depth (s/d) and upon the ratio $\dfrac{\text{actual shearing stress}}{\text{critical shearing stress}} \left(\dfrac{\tau}{\tau_{cr}} \right)$

See Fig. 17.

η_c = modulus factor in compression as given in Fig. 16 [23].

Local distribution of stress under point load.

The above formulæ are applicable to girders carrying stationary loads where bearing stiffeners are placed under each load. In a gantry girder, however, the wheel loads move over the span and therefore the load will not always be over stiffeners. When the wheel loads are between stiffeners local concentrations of stress will occur in the web, and the following formulæ have been derived to allow for these concentrations. (2),(24),(25),(26).

Direct horizontal stress under apoint load = σ_x.

$$\text{where } \sigma_x = \left(M - \frac{Pd_1}{2\pi} \right) \left(\frac{\frac{d_1}{2} + V}{I_{xx}} \right) + \frac{P}{\pi A} \quad \ldots\ldots (4)$$

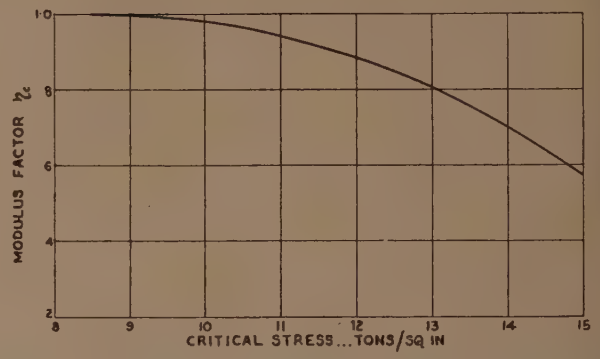


Fig. 16.—Modulus factor for compression, structural steel

Direct vertical stress under a point lead = $\sigma_y - \dfrac{2P}{\pi t V}$ . . . . . (5)

Shear stress under a point load = $\tau_{xy} = \tau_s + \dfrac{P}{2d_1 t}$ . . . . . (6)

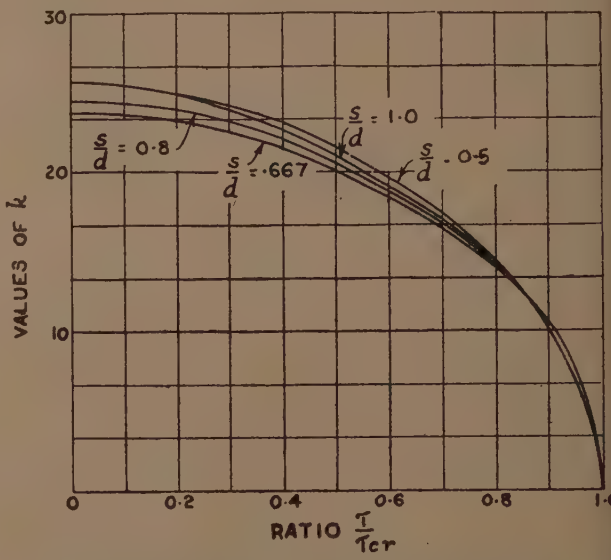


Fig. 17.—Values of factor k in rectangular plate subjected to bending and shear

Where M = total bending moment at the section due to all the wheel loads and the dead weight of the girder.
P = total wheel load over the section considered.
d_1 = total depth of girder.
V = depth of rail, top flange and flange angles.
I_{xx} = moment of inertia of the girder at the section considered.
A = cross-sectional area of girder at section.

t = thickness of web at section.
τ_s = shear stress in the web at the junction of the web and flange angles due to the dead-weight of the girder and the wheel loads, but excluding wheel load P.

These equations are in agreement with experiments already made but other experiments are needed to determine the spreading effect of various types of rails.

The principal compressive and shear stresses are calculated by the usual method.

$$\text{principal compressive stress} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right) + (\tau_{xy})^2} \quad . \quad . \quad . \quad (7)$$

$$\text{principal shear stress} = \sqrt{\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + (\tau_{xy})^2} \quad . \quad (8)$$

In all cases the principal stresses should not exceed the maximum allowable working stresses of the material from which the web is constructed.

Minimum Web Thickness of Sections at Mid Span

At various points along the girder, pure bending may occur as the wheels move over the span, and at these points the minimum web thickness can be calculated. Assuming the maximum compressive stress in the web is 9.0 tons per sq. in. approx. (corresponding to a maximum compressive stress in the flange of 9.5 tons per sq. in.), and allowing a factor of safety of 1.5, which has been shown by tests (19),(20),(21) to be adequate, the maximum permissible ratio of depth (d) to thickness (t) of web is found to be 130 approx. It will be seen that this differs from the minimum of 1/170 given in the existing specifications quoted earlier.

Maximum Web Thickness at End of Span

At the ends of the girder, the condition of pure shear largely exists, and by using the equation for this condition and adopting a factor of safety of 1.5, the maximum working stress in the web τ is found and is

$$\tau = (208)^2\left\{1 + \tfrac{3}{4}(s/d)^2\right\} \eta_s \left(\frac{t}{s}\right)^2 \text{ tons per sq. in.}$$

this equation is similar to the equation given in B.S.449 (48).

To use this equation, choose the thickness of web (t), so that the maximum working stress τ is equal to the safe working shear stress and determine the stiffener spacing (s), this will ensure that buckling will not occur.

Web Thickness in any Panel

The maximum thickness of web will usually occur at the ends of the girder and for small spans will be constant throughout. For large spans it will be more economical to reduce the thickness over the centre portion, in which case, calculations need to be made to find the point where the reduction can be located. The required web thickness in each panel (i.e., between stiffeners) can be found[2] by means of equations (1), (2) and (3), using the maximum values of the shear and bending stresses produced in the panel as the loads move over the span. Equations (4), (5), (6), (7) and (8) should be used to check that, with this web thickness, the local concentrations of stress in the web are not excessive.

Position of Web Joints

The vertical web joints should be placed at the point of minimum vertical shear. For simply supported girders, zero shear due to self-weight occurs at the centre of the span. In the case of shear from the crane wheel loads, this is zero at practically every point along the girder as the wheel loads move over the span. Another criterion therefore has to be considered and that is the point which has the lowest maximum shear, and this is the centre of the girder. From considerations of shear web joints should accordingly be placed at the centre of the girder. A further advantage of such an arrangement is that gantry girders are usually symmetrical about the centre and hence the position of the joint will usually coincide with a stiffener. Web joints should always be placed under or close to a stiffener so that they will not weaken the stability of the web against buckling.

For spans over 70 ft., where difficulty may be encountered in transporting the girder as a single unit, the web splice may have to be moved from the ideal position in the centre to coincide with the flange splices, thus splitting the girder into two or more transportable sections. In this case it is much better to alter the position of the web, rather than the flange joints, since the maximum shear at any section follows a linear law, while the maximum bending moment follows a parabolic law.

Should the thickness of the web over the central portion be reduced, as has already been explained, the webs joints will obviously have to be moved from the ideal position at the centre to that position where the thickness changes.

Since the maximum width of plate is limited by the size of the mill, it will not usually be found feasible to construct the webs of larger girders in one piece and a horizontal web joint will have to be incorporated. It may in this case be found more economical on large span girders to construct the central portion of the web of thinner section employing two welds, rather than to join two equal thicknesses at the centre. This construction is justified since the centre of the web is under low bending stress and does not contribute any great resistance to buckling. Web shear stresses will have to be carefully considered in such a case however.

Web Stiffeners

Introduction

As long as the local concentrations of stress in the web due to the wheel loads are below the allowable stress, the web can be prevented from buckling by a suitable arrangement of web stiffeners. If the bending and shear stresses in the web are too high, the stiffeners will need to be placed so close that they become uneconomic, and then it is better to increase the thickness of the web. If the web is subjected to pure bending, vertical stiffeners will have little effect on the stability, and recourse must be had to horizontal stiffeners to prevent buckling under high stresses. The loading conditions are accordingly more exacting than in the case of girders carrying only concentrated static loads under which bearing stiffeners can be placed so as to distribute the load over the whole depth of the girder.

Stiffener Spacing

Existing regulations in different countries vary considerably (Fig. 18) but by using the theory of stability, formulæ for suitable stiffener spacing have been developed[2].

At the extreme end of the girder where the web is largely subjected to pure shear, the required stiffener spacing, s, is given by :—

$$s = \sqrt{\frac{5{\cdot}35\eta_s}{\frac{1{\cdot}5\tau}{12{,}105} - \frac{4\eta_s t^2}{d^2}}}\,xt \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Over the centre portion of the girder, where the web is subjected to both shear and bending, the required stiffener spacing s, is given by :—

$$s = \sqrt{\frac{5{\cdot}35\eta_s}{\frac{1{\cdot}5\tau 12(1-\mu^2)}{\pi^2 E\ \theta} - \frac{4\eta_s t^2}{d^2}}}\,xt \quad . \quad . \quad . \quad . \quad . \quad (2)$$

where $s < d$

$$s = \sqrt{\frac{4\eta_s}{\frac{1{\cdot}5\tau 12(1-\mu^2)}{\pi^2\ EQ} - \frac{5{\cdot}35\eta_s t^2}{d^2}}}\,xt \quad . \quad . \quad . \quad . \quad (3)$$

where $s > d$

$$\frac{k}{k_1} = \frac{1{\cdot}5\sigma_x}{\eta_c} \frac{12(1-\mu^2)d^2}{23{\cdot}9\ \pi^2 E\ t^2} \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

where s = required stiffener spacing in inches.
θ = the value of the ratio τ/τ_{cr} as given by Table I for the particular value of the ratio k/k_1.
σ_p = maximum compressive bending stress in the web in tons/sq. in.
η_c = modulus factor in compression.
d = unsupported depth of web in inches.
t = thickness of web in inches.

TABLE I—Reduction of k for Bending Stresses due to simultaneous action of shear

T/T_{cr}	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
% of k	0.99	0.97	0.94	0.90	0.84	0.77	0.68	0.57	0.41	0

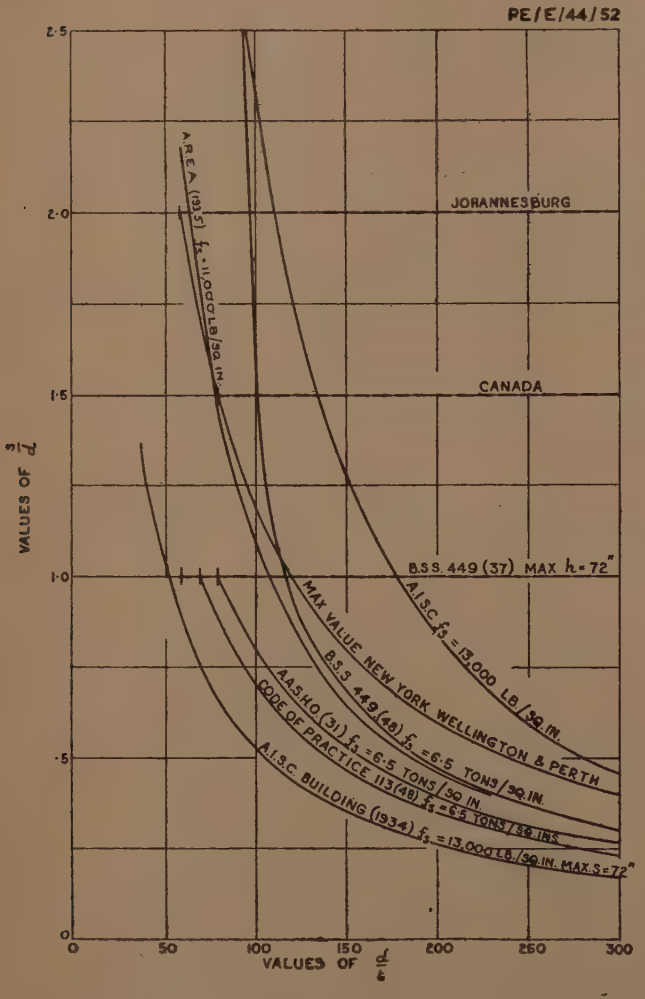


Fig. 18.—Stiffener spacing by various specifications

μ = Poisson's ratio for the material.
E = modulus of elasticity for the material.
τ = shear stress in web panel in tons/sq. in.
η_s = modulus ratio in shear.

Application to Design

In order to reduce the number of calculations which this method involves, the required ratio s/t has been calculated for varying values of d/t, bending stress and shearing stress and is shown plotted in Fig. 19. This figure can thus be used to find the necessary stiffener spacing for any girder, the procedure being as follows :— Having evaluated the slenderness ratio of the web (d/t) this value is found on the abscissa and a vertical line is projected up to cut the appropriate full line of extreme fibre bending stress in the web. A horizontal line is then projected across from this point to cut the appropriate dotted line of shear. A vertical line is then drawn to cut the web slenderness ratio line (chain dotted) and the required ratio $\frac{\text{stiffener spacing}}{\text{web thickness}}$ can be read off the right hand ordinate. A worked example is shown at the bottom of the figure.

The case of the stiffener spacing at the end of the girder, where there is no bending stress, can also be found from the figure by using the full line for zero bending stress. It will be seen in this case that with a shear stress of 6 tons/sq. in. slenderness ratios greater than 105 require a stiffener spacing less than the depth of the web (d) and ratios less than 105 require a spacing greater than the depth. The limit of stiffener spacing is reached at a web slenderness ratio of about 90, below which no stiffeners are required for stability, this being in agreement with previous experimental work[21]. Stiffeners are usually incorporated in practice, however

since they increase the torsional rigidity of the girder and help to keep it square during construction.

The figure also shows the limiting slenderness ratio for a particular bending stress and also the allowable combination of bending and shear stresses. Thus the limiting slenderness ratio for a bending stress of 10 tons/sq. in. is 103 and for a stress of 8 tons/sq. in. is 143. Also a web subjected to a bending stress of say 9 tons/sq. in. cannot also safely carry a shear of 6 tons/sq. in. for the range of web slenderness ratios shown. However, such a web can carry a shear stress of 5 tons/sq. in. if the slenderness ratio lies below 95.

An examination of existing girders shows that a certain proportion have web slenderness ratios of less than 90 so that the stiffeners do not ultimately strengthen the webs against buckling. As the shear increases towards the end of the girder, the shear stress increases so that the ratio of stiffener spacing to depth of girder should decrease and this is so in the girders with parallel flanges where the stiffener spacing decreases towards the ends. The majority of existing girders with sloping or curved flanges maintain a constant stiffener spacing over the last quarter of the span. Actually the spacing should decrease slightly towards the ends, although not to the same extent as for girders with parallel flanges. This can be seen from equation (1) for pure shear, which shows that for a constant stiffener spacing (s) a decrease in the depth increases the critical buckling stress, although not in a direct ratio. On the other hand, since the shear increases and the depth decreases towards the ends, the actual shear stress increases rapidly. In order, therefore, to maintain the same relationship between actual and crippling stress, the stiffeners must be moved closer together. To reduce the actual shear stress, the use of web cover plates at the ends of girders with sloping flanges is to be recommended.

Stiffener Strength

Besides preventing the web from buckling, web stiffeners support the compression flange against local buckling and increase the torsional rigidity of the girder. The strength of the stiffeners should be sufficient to form nodal lines at their location as and when the critical web stress is reached. Any increase in stiffener size beyond the minimum necessary to form nodal lines will not increase the web strength, because the web cannot be stronger than its weakest panel, which at this critical stress will have commenced to buckle.

From theory and tests (2),(22),(27)to(32) the necessary strength can be found from :—

$$I_s = qdt^3 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

where q = a factor dependent upon the ratio s/d. Values of q being given in Table II and Fig. 20.

d = unsupported depth of web in inches.

t = thickness of web in inches.

I_s = moment of inertia of stiffeners about the centre of the web if double stiffeners are employed, and about the inner face of the web if the stiffener is only on one side of the plate.

This equation does not give an idea of the shape of stiffener to be used but from experience, a stiffener, without any kind of flange at its outer edge, should not be thinner than a sixteenth part of the width or one-hundred and fiftieth part of the length of the stiffener whichever is greater. Wherever possible, stiffeners should be constructed in tee form with the flange placed outwards from the web, so as to stiffen the outer edge of the stiffener. The best form of stiffener will have the largest radius of gyration for a given cross-sectional area, the largest part of which will be as far as possible from the web. A stiffener on one side of the web will accordingly be more efficient than two smaller stiffeners one on each side. If this arrangement is adopted, attention must be given to supporting the compression flange against local buckling, so that on the side without the stiffener, a triangular gusset between web and flange may be necessary. Also to avoid too much asymmetry, the single stiffeners will require to be placed on alternate sides of the girder. As the least material to connect the stiffener to the web is attained by the use of welding, welded stiffeners are to be recommended, even where the remainder of the girder is riveted. As stiffeners in crane gantry girders merely support the web, their type, unlike bearing stiffeners, is not limited, so that they may be joggled or straight. To prevent twisting of the flange about the web, it is essential for the top of the stiffener to be fixed or packed flush with the underside of the compression flange. The bottom of the stiffener should not be welded to the tension flange in such a way that the weld runs across the flange, but horizontally at the extreme edges of the flange.

The foregoing remarks do not apply to end stiffeners which are essentially bearing stiffeners. End stiffeners should always be in pairs, symmetrical about the webs, and should project to the outer edges of the flange. They should be designed as a column of effective length equal to three-quarters of the length of the stiffener, while the bearing stress on that part of their cross-section which is clear of the root of the flange angles should not exceed 12 ton/sq. in. for mild steel.

It has been shown[2] that vertical stiffeners do not materially affect the resistance to buckling of a web panel subjected to bending stresses. The web can be increased to this end but this is at the expense of added weight. A more effective method is to use horizontal stiffeners in the compression area of the web. Recommendations on horizontal stiffeners however need further consideration.

TABLE II.—Factor "q" for values of Ratio s/d

s/d	0.5	0.55	0.6	0.65	0.70	0.75	0.80	0.85	0.9	0.95	1.0
Average value of q	5.55	4.15	3.15	2.638	1.926	1.55	1.276	1.062	0.9	0.756	.0626

End Connections and Allowance for Expansion

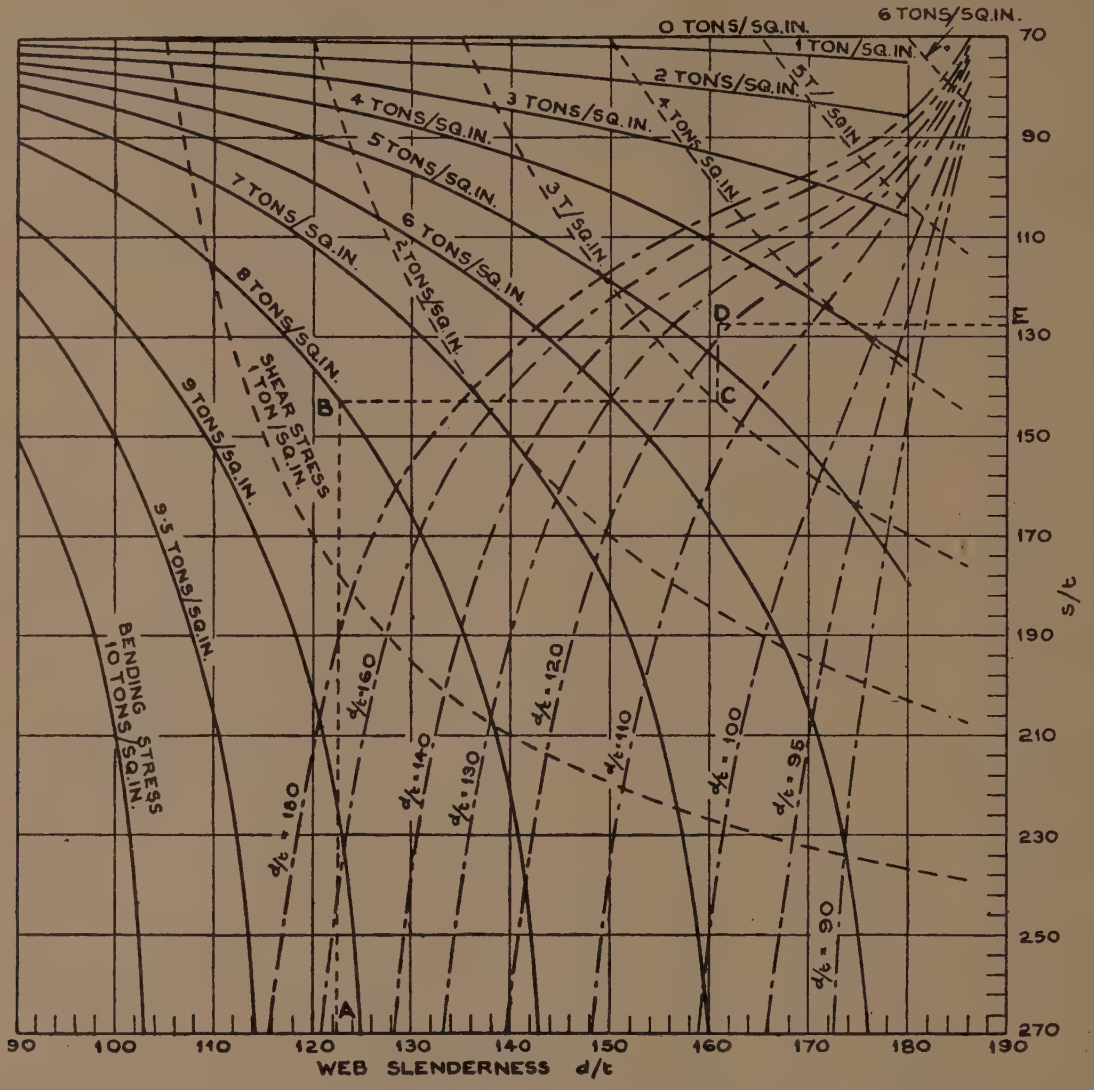
The bearing plate must spread the load and spread it uniformly. The more end stiffeners and webs there are, the less spreading, and hence thicknesses of bearing plate, is necessary. To avoid uneven bearing pressure, provision, as far as possible, is needed to allow the bottom flange to extend and tilt. In spans over 50 ft. some form of pinned end might have been considered advisable although such provision has not been made except in the latest building having spans over 100 ft.

Even so, one span of 117 ft. has been built without any special provision for rotation of the bearings.

Although in competitive work, holding down bolts in base plates to structures are omitted, holding down bolts in bearing plates to girders must be provided even in free spans, although uplift is not to be expected. Positioning laterally, longitudinal continuity and resistance

surge girder is provided. A more elaborate way than cross bracing between girders or plating to the leg of the roof stanchion is the use of separate frames in the form of an " A " which permit differential deflection between adjacent girders.

In buildings, the range of temperature to be allowed for is less than for bridges where the usual provision



EXAMPLE.:- A WEB, SLENDERNESS RATIO 122·5, IS SUBJECTED TO A BENDING STRESS OF 8 TONS/SQ.IN. AND A SHEAR STRESS OF 3 TONS/SQ.IN. STARTING FROM THE VALUE OF SLENDERNESS RATIO (POINT A) FOLLOW VERTICAL DOTTED LINE UNTIL IT CUTS FULL LINE REPRESENTING A BENDING STRESS OF 8 TONS/SQ.IN. (POINT B) ACROSS HORIZONTALLY CUTTING SHEAR LINE FOR 3 TONS/SQ.IN (POINT C), UP TO THE INTERPOLATED VALUE OF THE SLENDERNESS RATIO (POINT D) FINALLY ACROSS TO POINT E GIVING THE VALUE OF THE STIFFENER SPACING AS 128 x t.

Fig. 19.—Chart for computing stiffener spacing

to tractive forces, expansion and vibration is also maintained by longitudinal bolts through the end plates. The bolts should screw down on sleeves or ferrules over which the adjoining end plate may slide to allow for expansion. These bolts should be arranged in groups in the centre of the depth of the girders and not uniformly over the depth as in many existing girders.

Lateral surge from the cranes is transferred largely through the top flange at the end of the span unless a

is from ¾ to 1 in. for every 100 ft. From experienc the use of sliding plates or steel rollers as used in bridg can be dispensed with because the stanchion must b very stiff and well anchored at the base if it is to off the necessary resistance to make sliding of the girde possible.

An effect somewhat similar to temperature is th tendency of " built-up " riveted steelwork to increase length during erection. As an example of this, bear which butt into webs and on to flanges are usual

made $\frac{1}{8}$ in. and $\frac{1}{4}$ in. less respectively than the theoretical net dimension, the larger $\frac{1}{4}$ in. dimension allowing for oversized rolled sections. Where stools are used on stanchions, the centres of the holes are drilled to the net dimension. The net dimension is also used for the hole centres of riveted or welded crane girders in which the holes for attachment to the stanchion caps are marked out and drilled after the girder has been completed.

Camber

The provision of an initial set upwards during fabrication may be beneficial in improving appearance and drainage, in producing horizontal flange plates under load as assumed in design, in avoiding a sagging effect at the centre due to the greater thickness of flange

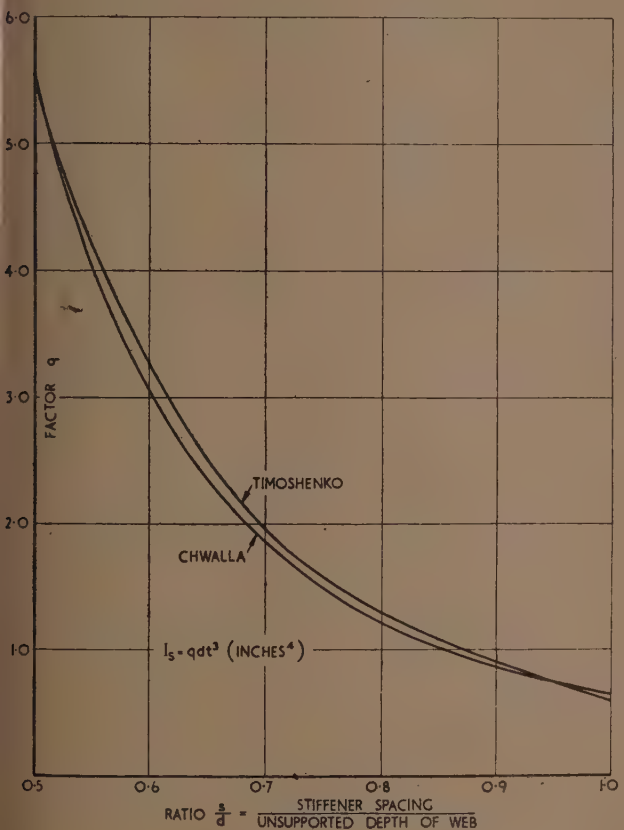


Fig. 20.—Factor q for calculating stiffener strength

plates, in allowing for creep in the metal under load, and in reducing the gradients to be negotiated by the crane.

In heavy gantry girders, especially if welded, the difficulties of providing a camber during fabrication tend to outweigh the advantages. The principal consideration is to maintain the level of the track as far as possible. Without camber, the crane has to climb a gradient when moving towards the end of the span although as the crane travels further from the centre the deflection and hence the gradients are reduced. With camber, the crane enters its maximum gradient when it first rolls on to the girder. To balance these two gradients, the camber provided should be equal to the dead load deflection plus half the live load deflection. Deflections should preferably not exceed 1/1000 of the span, and usually are not more than 1/500 giving a maximum gradient of about 1 in 200.

As the girder spans over the furnaces in a melting shop are often longer than the girder spans on the other side of the shop, the deflections of the supporting girders at each end of the crane may differ, causing the crane to skew during long travel, so giving rise to flange wear[32]; this condition is important and should not be overlooked. One other difficulty is the use of asymmetrical girders, such as girders with surge girders attached or two parallel girders having differing moments of inertia when braced together. These composite girders tend to twist under the load and accordingly need careful consideration.

Loading

In addition to the self weight of the girder and weight of crane and load, allowance must be made for the dynamic effect due to the crane and load. To this end, a percentage is either added to the static crane load and the usual working stresses are adopted or alternatively the static loads are used and the working stresses are reduced. Both methods have the effect of maintaining the safety factor between the working stress and the yield stress of the material during high transient stresses. As the dynamic effect is dependent on the relative masses of the crane and girder, the correct impact factor differs in each case.

Values of factors for vertical impact, lateral and longitudinal surge as recommended by various authorities are shown in Table III and these are seen to differ appreciably in some cases.

Most values are stated to be the minimum, so that the correct value is open to some doubt.

Vertical impact covers vibration, jerk due to sudden lifting of the load, possible slip of slings and irregularities in rail surface and the average value of 25 per cent. appears adequate. For longitudinal surge, the coefficient of friction between steel and steel is 0.2 but the crane may oscillate about a line perpendicular to the rails causing flange and rail contact and the evidence of flange wear indicates that a factor of 5 per cent. of the wheel loads is possibly too low. For lateral surge, a percentage of the weight of crab and lifted load is mostly required. As the crab and load is often traversed during long travel lateral and longitudinal surge should be considered simultaneously, although this is not insisted upon in the British Standard.

It is not wholly certain that the action of a dynamic load is similar to a static load at the same point. It is usually assumed that the stresses are similar but it might be better to assume a similar elastic shape deformation in the structure. The objection to such a method is that any error in the assumed shape is magnified by the successive differentiation which is required to link deflection and stress. The most accurate way of relating the actual stresses to the known applied loads is to carry out stress measurement on existing girders and this is accordingly being pursued.

Until such test results are available it is suggested that the following value for impact loading should be assumed for iron and steelworks cranes :—

Vertical impact loading = 25% of maximum wheel loads.

Longitudinal surge = 10% of maximum wheel loads.

Lateral or " side " surge
(a) Charger cranes = 25% of weight of crab and lifted load.

(b) Magnet cranes = 20% of weight of crab and lifted load.

(c) Casting cranes = 10% of weight of crab and lifted load.

The loading intensity recommended is :—

Crane Type	Loading		
		Vertical	Lateral Surge and Longitudinal
Charger Magnet Casting	two cranes running buffer to buffer	both cranes 100% loaded one crane 100% loaded one crane unloaded	one crane 100% lateral loading one crane 50% lateral loading

Conclusions and Recommendations

In this paper, which is based on two more detailed papers([2]) and ([3]), it has been shown how improved design and economy can be achieved by closer adherence t fundamental theory and to the best practice and th main conclusions may be summarised as follows :—

The type of girder most suited to crane gantry girder in steelworks is a single webbed plate girder and it depth should preferably be as defined by the equatio given in the text and within the range of 1/14th t 1/8th of the span.

Many gantry girders are more slender in section tha British Standard beams and the usual equations fo permissible stress in the flanges are not necessaril applicable. A practical means of applying the funda mental equation in the design office is described and thi enables the stockier sections in particular to be use over a greater range of slenderness ratios, and for th more slender sections over a more restricted range tha is suggested by B.S.449. Graphs have been develope to test given sections for permissible stress and also t select suitable sections directly. Flange splices, whe required in the larger girders should, as a compromis between theory, fabrication, transport and erectio considerations, be arranged at quarter to fifth spa points, subject to other considerations such as maximu available lengths of plate.

TABLE III.—Allowance for Impact as given by Various Specifications

Author	(a) Vertical Impact Percentage Increase	(b) Longitudinal Surge	(c) Lateral Surge	Remarks
The Institution of Structural Engineers Report on Loads and stresses for crane gantry girders (1927)	25% of the maximum wheel loads	Not less than 5% of the maximum wheel loads	Not less than 5% of the weight of crab and lifted load divided equally between the rails	All loads to be acting at th same time, with two crane running bumper to bumpe the minimum vertical impac shall be 15% of total whe loads
British Standard No. 449 (48) The use of structural steel in buildings	25% of maximum wheel loads	5% of maximum wheel loads	10% of the weight of crab and lifted loads	Forces (b) and (c) shall not b taken as acting at the sam time. The permissible benc ing stress can be increased b 10% for combination of loac (a) and (b)
American Institute of Steel Construction Specifications, 1946	25% of wheel loads	10% of maximum wheel loads unless otherwise stated	20% of the weight of crab and lifted loads	
Russell & Dowell, "Competitive Design of Steel Structures"	25% of wheel loads	Not less than 10% of maximum wheel load	10% of lifted load	
Redpath Brown & Co., Ltd.	20% of wheel loads	Not less than 5% of the maximum wheel loads	Not less than 5% of weight of crab and lifted load	
Schulze B.I.O.S. Report. Final Report No. 1822		7½% of maximum wheel loads	10% of maximum wheel loads	
Dr. Witt B.I.O.S. Report. Final Report No. 1822	A percentage of live load bending moment diagram	10% of maximum wheel loads	15% of weight of crab and lifted load	

The minimum web thickness should not be less than $\frac{1}{130}$-th of the unsupported depth of the web when the compression flange stress is at a maximum of 9.5 tons per sq. in. (c.f. the figure of $\frac{1}{170}$ in existing specifications). The best thickness will be attained when, at some higher loading, the web will have reached the yield stress at the same time as it commences to buckle. Stress concentrations under wheel loads should be examined and equations for this are given, and in this connection, the effect of rail "spread" needs to be further examined experimentally. Vertical web joints should preferably be located at the centre, except where, as in large spans, it is economical to reduce the web thickness over the central portion.

With the exception of those at the ends, web stiffeners in gantry girders are required to prevent the web from buckling, to support the top flange against local buckling and to increase the torsional rigidity of the girder. They should accordingly be designed essentially as "intermediate" stiffeners and equations for their spacing and strength are given in the text. Attention should be given to the most efficient type of stiffener to maintain minimum weight. Horizontal web stiffeners enhance the resistance of the web to bending stresses and they are recommended, but the estimation of their strength needs further consideration.

From experience on existing girders, no special arrangement is required for end connections and girder seatings up to spans about 100 ft., unless the girders are continuous or rigidly tied to another twin or to a surge girder, when there is a tendency to twist and this has to be restrained at the bearings. Some girders with spans over 100 ft. have been articulated at the cross bracing to eliminate twist, frames have been provided at the bearings to resist the lateral forces and pins have been inserted to assist rotation at the bearings due to deflection, while other girders of similar span have been built without these precautions. In view of this difference in practice the subject needs further examination to arrive at the best and cheapest detail to cater for temperature, differential deflection, the transmission of surge and allowance for rotation of the bearings.

The adoption of camber helps to limit the gradients for the crane wheels, to avoid a sagging effect due to extra thickness of plates at the centre and to make the flanges horizontal when under load as assumed in design. If used, camber should be equal to the dead-load deflection plus half the live load deflection at the centre. When determining camber, the deflections of the two gantry girders at each end of the crane should be equalised as far as possible to limit the tendency of the crane to slew and also the torsional effect of the surge girders should be taken into account. In large girders, the cost of fabrication to include camber probably outweighs the advantages.

The existing method of allowing for the dynamic effect of the wheel loads by increasing the static load by fixed percentages does not take sufficient account of the relative masses of the lifted load, the crane and the girder, so that experiments on existing girders are recommended to relate induced stresses to actual loads. Until these factors have been established, impact allowances given in the text which are considered to be suitable, may be used.

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Plate 1—General view. Left foreground: residential quarters. Right: power station site and native compound
By courtesy of "Sky Fotos"

The Structural Aspect of Power Station Design in South Africa*

By G. M. Frost, M.I.Struct.E., M.(S.A.)I.C.E., F.R.S.A.

Summary

This paper sets out the results of an investigation carried out by the Structural Engineering department of ESCOM to determine, if possible, the most economical design for steel frame power station buildings in South Africa.

The Electricity Supply Commission of South Africa, commonly known as "Escom," is a Corporate body created by the Electricity Act of 1922, and in terms of Clause 10(4) of the Act :—

"It shall be a general principle of the Commission that its Undertakings shall, as far as practicable, be carried on neither at a profit nor at a loss, and that its charges shall be adjusted accordingly from time to time."

It is therefore obvious that Escom in common with similar bodies in other countries, is vitally interested in, among other things, the economic design of the buildings housing its power station plant.

To this end the Structural Engineering department of Escom has for some time past been investigating the comparative costs of various designs of power station main buildings, and the object of this paper is to lay before you the results of those investigations and the conclusions which have been reached.

It is essential, in this connection, to take account of South African conditions, as these affect both cost and design.

To begin with, the Union of South Africa is a large country, actually about 5½ times as large as the United Kingdom, and the greater part is industrially undeveloped and very sparsely populated. Industrial areas are, except for the gold, coal and base metal mines, concentrated in or around the large towns.

The principal and secondary towns are now connected by rail and first class tarred roads, but, generally speaking, beyond that the roads range from good to very bad, with a surface of anything from hard-packed sandy soil to plain clay, which become virtually impassable after heavy rains, so that transport of material to site may be,

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ıd often is, a major and expensive problem, adding ›preciably to the finished cost of the structure.

The major portion of the present industrial expansion the Union is taking place in the Orange Free State and ıe Transvaal with the result that our latest stations in ıese provinces are sited away in the veld with the ·arest dorp 20 or 30 miles away. For this reason, ther Escom or the contractors must provide living ccommodation for their employees, both white and lack, on the site. Moreover such accommodation, wing to the present day difficulties in attracting labour › such a site, has to be of a standard which would have ·en described, 25 years ago, as positively luxurious. or this Escom finally pays and the cost of the accomıodation and amenities go to swell the cost of the nished job.

design of Vierfontein as a 150 MW station it was increased to 210 MW, and before the first few bays of the turbine house had been erected it was again increased to 300 MW. In fact, broadly speaking, if we were prepared to reduce the life of the station by using the available coal more rapidly, there would be no reason, apart from "handleability," which is referred to in more detail later, why Vierfontein station should not be increased to 600 MW or more.

On the face of it this should present no difficulty, but reference to Plate 1, which shows the site of Vierfontein Power Station soon after work was commenced, will show why, with no precise information regarding the final extent of the station and the first machine required to be running as soon as possible, work must be commenced at one end of the station and the other end must

Plate 2—Vierfontein Power Station under construction

One would imagine that in building in the veld the lmost unlimited space available would be an advantage, ut strangely enough this can be something of a handicap.

The reason is this—in the United Kingdom it seems, hen building a new station you are usually presented ith a limited site; it may be large or small but it is lefined as to position and area. This limitation, ogether with your more precise knowledge of the inended capacity of the station, makes it possible for you o lay out your station at once, complete with any ossible future extensions.

We, in South Africa, on the other hand are not limited y area, only, in general, by economic considerations uch as the availability of coal and water and by what I ıay call the "handleability" of the station. Because f the unreliability of load forecasts due to the very apid industrial growth of the country in recent years, oupled with the availability of unrestricted site areas djacent to large coal fields and adequate water supplies, ıakes it a rare occurrence for us to know, when we ommence the design of a station, what the final capacity ill be. For example, before we had completed the

be left free of any obstruction whatever to allow for any possible future extensions, though these may not be undertaken for years or perhaps ever.

This means that one end of the site is completely useless in so far as the siting of such ancillary buildings and structures as the Administrative office block, switch operating block, workshops and stores, H.T. yard, cooling towers, water purification plant, railway sidings, roads, etc., are concerned.

For example, many British stations are provided with two handsome reinforced concrete chimneys, one at each end of the station, which from an aesthetic point of view is a very pleasing and often economically sound arrangement. Our stations are mostly away in the veld so we are not so concerned with architectural appearance, nevertheless we do want a certain amount of good looks, but we are limited in scope because the only place available for the chimneys is down one side of the boiler house.

For the same reasons practically the only location for the H.T. yard and cooling towers is down one side of the turbine house, which leaves only the area at what we call

the permanent end of the main buildings for such things as the office block and stores and workshops.

It may be said that these things only place a limit on the possibilities of layout and so have little or no effect on the cost of the station. But consider for a moment the effect that these limitations have on such things as, for instance, the design of the storm water and sewerage systems.

The storm water system has to collect and dispose of the rain water from all the roofs of the buildings and from the levelled areas around the station, the quantity ranging up to 5 in. of rain per hour. This system also handles the drain water from the turbines, heaters, and steam receivers, and the cooling tower ponds blow-down.

In order to collect all this water the system must generally be composed of a number of parallel drains running the length of the station, both inside and outside the turbine and boiler houses, brought together at one end of the station by means of a header pipe leading to the final outfall.

In order to collect the longitudinal drains the header must pass transversely across the station at one end or the other. The longitudinal drains must be graded from just below ground level at one end to a depth determined by their length and grade at the other. But there lies the trouble. The drainage system is one of the first things to be designed and constructed, but at that time we do not know what the full extent of the station will be, so we can neither determine the length of the main buildings and the longitudinal drains nor, consequently, the depth of the header.

Moreover we can only place the header at the permanent end of the station, for fear that if we placed it at the extension end of the station the extension would over-run it at some future date.

There is only one way out of this dilemma and that is to assume that the station will eventually be extended to a " handleable " maximum, which we have fixed, from past experience, at 12 sets placed transversely.

Precisely the same remarks apply of course to the sewerage system, which must collect and dispose of the sewage from offices and internal change rooms, the future positions of which are unknown when the layout of the reticulation is being planned.

It is obvious that if the station is never extended to 12 sets a great deal of costly excavation will have been undertaken to no purpose, but under the circumstances, unavoidably.

That is the background against which the following remarks must be viewed.

Table 1 was prepared to show the comparison, one with another of the cubic contents and steel required, between five South African stations and compared with the same data for 13 British stations.

Pretoria " B " which is not an Escom station, is a reinforced concrete structure and was added to Table 1 in order to compare that type of structure with an all steel structure which is the type of all Escom stations, up to date.

As can be seen Columns 2 and 3 merely describe the plant to be housed. Columns 4, 5, 6, 7 and 8 show the total volume and volume of various parts of the housing provided at the different stations. The interest of these figures lies in the wide variation shown in the volume per kW provided in the South African stations and, by comparison, much the same variation in the volumes provided in the British stations.

It should be, without question, the object of every designer of power station buildings to provide as much space as he can for the least possible cost per kW installed.

Columns 9, 10 and 11 are of interest as showing the total amounts of steel required, the tons per megawatt and the lb. per cu. ft. of volume. Here again a wide variation will be noticed. It should be noted that in the case of the South African stations such things as hand-railing, chequer plating, open grid flooring, roof sheeting (where that is used), stairways and treads, have been included in the weight of steel. I am unfortunately unable to say whether this is so in the case of the British stations.

Columns 12 and 13 have been included because of the difference in design between the turbine house and boiler house, at least in so far as Vierfontein and Taaibos are concerned, and they show the tons of steel required per MW in the turbine house and the tons of steel per 1,000lb. of steam in the boiler house for each station.

Columns 14 and 15 show, respectively, the cost of the steel frame buildings per cu. ft. of volume and per kW installed. This information regarding British stations I have been unable to obtain.

Before attempting to reach any conclusions from a study of Table 1 it must be remembered that a great many factors have to be considered when planning a power station. Such things as the length and run of H.P. pipework, the length and runs of cabling, the layout of C.W. pipes, the disposition of the turbo-generators and boilers, etc., must be considered, all in relation to one another and their individual and combined effect on the plan, elevations and sections of the buildings, so that the station requiring the least amount of steel per cu. ft. or even per kW is not necessarily the most economic when viewed in the light of the total cost of the station as a whole.

For these reasons it is impossible to dogmatise and is very difficult to generalise, so that in comparing these various stations we must take into consideration the restrictions imposed by the requirements of the plant and location, in each case.

The best we can do therefore is to consider each station in the light of its own particular circumstances.

Let us take them then in descending order of tons of steel required per megawatt of installed capacity.

Umgeni Power Station is in Natal, in a peri-urban area adjacent to and overlooked by several small residential areas, and was subject to the building by-laws of Durban and the Durban Elevational Control Committee's requirements. For these reasons flat roofs were adopted and architectural considerations affected the structure much more than at Taaibos, Vierfontein or Wilge. For the same reasons the flat roofs were constructed of hollow pre-cast concrete beams. In order to produce a pleasing clean cut internal appearance and to eliminate all obstructions to pipe and cable runs, box section rigid portal frame construction was used throughout, and no longitudinal or transverse vertical bracing was allowed.

The generators at Umgeni are disposed end to end longitudinally and there is a single row of boilers. To keep the boiler faces clear the boiler house columns were spaced at 48 ft. 0 in. centres and the turbine house columns lined up with them, resulting in long span crane eaves and roof girders. The 75 ton crane has a span of 51 ft. 0 in.

The rigid portal type of construction combined with the absence of all vertical bracing resulted in some very large bending moments in the columns in both directions and in the connecting beams.

The concrete roofs resulted generally in a dead and superimposed load of approximately 112 lb. per sq. ft. in addition to which the boiler house roof carries the F.D. fans and withstands their vibrations.

TABLE I

SOUTH AFRICAN STATIONS

Station	Installed Capacity		Total volume of buildings × 1000 cu. ft.	Volume of buildings cu. ft. per kW installed capacity				Quantity of Steel					Cost per cu. ft.	Cost per kW installed
	T.A. sets megawatts	Boilers lb. per hour × 1000		Total building	Turbine house	Boiler house	Switch house	Short tons	Tons per megawatt	lb. per cu. ft.	Turbine house tons per megawatt	Boiler house tons per 1000 lb. steam		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Umgeni ...	L 3×30	S 6×180	4,365	47.8	11.4	36.4	—	5,160	57.3	2.36	18.7	3.21 (4.54)	£0.107	£5.6
Hex River ...	L 3×20	S 4×200	3,012	50.2	16.4	33.7	—	2,884	48.0	1.95	21.0	2.03 (3.23)	£ .087	£4.36
Vierfontein	T10×30	D17×210	16,431	54.7	17.5	37.2	—	8,630	28.7	1.05	9.1	1.56 (2.19)	£ .052	£2.87
Taaibos ...	T 7×60	S 7×580	17,214	41.0	15.3	25.1	.53	12,000	28.5	1.4	7.5	2.17	£ .07	£2.86
Wilge No. 1	L 2×30	S 4×150*	877	14.6	14.6	—	—	680	11.3	1.55	11.3	—	—	—
**Pretoria "B"	T 3×30	D 6×210	4,274	47.5	19.5	28.0	—	SS.750 RS.820	17.5	0.74	—	—	£ .045	£2.12

*Outdoor boilers. **Reinforced concrete frame.

BRITISH STATIONS

Station	Installed Capacity		Total volume of buildings × 1000 cu. ft.	Volume of buildings cu. ft. per kW installed capacity				Quantity of Steel			Cost per cu. ft.	Cost per kW installed	
	T.A. sets megawatts	Boilers lb. per hour × 1000		Total building	Turbine house	Boiler house	Switch house	Short tons	Tons per megawatt	lb. per cu. ft.			
Brunswick Wharf, Poplar ...	L6×52.5	S12×320	18.025	57.2	21.1	36.1		16.267	51.7	1.8			L = machines disposed longitudinally.
Croydon ...	L4×52.5	8×320	13.614	64.8	21.2	42.8	0.8	9.441	45.0	1.38			S = boilers in single row.
Braehead ...	L4×50	8×300	11.103	46.7	19.7	27.0	—	8.176	40.88	1.47			T = machines disposed transversely.
Straythorpe ...	2×60	6×240	5.094	51.0	20.3	30.7	—	3.853	32.1	1.51			D = boilers in double row.
Bankside ...	2×60	4×375	13.775	65.1	32.2	34.8	7.1	5.667	47.2	0.82			
Little Barford	4×30	5×300	4.362	35.4	15.6	19.8	—	2.352	19.6				
Cliff Quay ...	L6×45	S9×365	10.880	40.4	18.3	22.1	—	6.720	24.8	1.23			
Deptford West	T 215	D	9.469	44.1	16.1	22.9	5.03	7.998	37.2	1.68			
Battersea "A"	L 228	S	12.612	55.4	20.5	24.2	10.75	17.662	77.4	2.8			
Battersea "B"	L 282	S	15.493	55.0	18.2	28.1	8.7	14.717	52.2	1.9			
Dunston ...	200	—	9.670	48.35	16.13	29.06	3.16	—	—	—			
Fulham ...	T 300	—	18,361	61.2	16.2	39.2	5.8	—	—	—			
Hams Hall "B"	L 300	—	20.601	68.7	15.5	46.5	6.7	—	—	—			

See Proceedings of the Joint Engineering Conference 1951, part 4, POWER. E.S.C. Library Index No. 621. 31213/20.

All these things resulted in a structure which, for reasons not under the structural designer's control, proved to be a very heavy one.

We consider, 57.3 short tons per MW is very heavy, especially as the turbine house does not include an off-loading bay.

Hex River Power Station is close to the town of Worcester and is another in which the generators are disposed end to end longitudinally and the boilers are in a single row.

The structural design of this station is the same as tha of Umgeni except that above firing floor level vertica bracing has been introduced between the bunke columns; the only flat concrete roofs are over the bunke and auxiliary bays, both of which carry heavy tanks the remaining roofs are sloping and covered with pro tected metal V-beam sheeting. In addition the bunke columns at this station are spaced at 53 ft. 0 in. centres but the main building and crane columns are at a third o this distance, giving crane, eaves and roof girders onl 17 ft. 8 in. long. The 56 ton crane has a span of 44 ft. 0 in

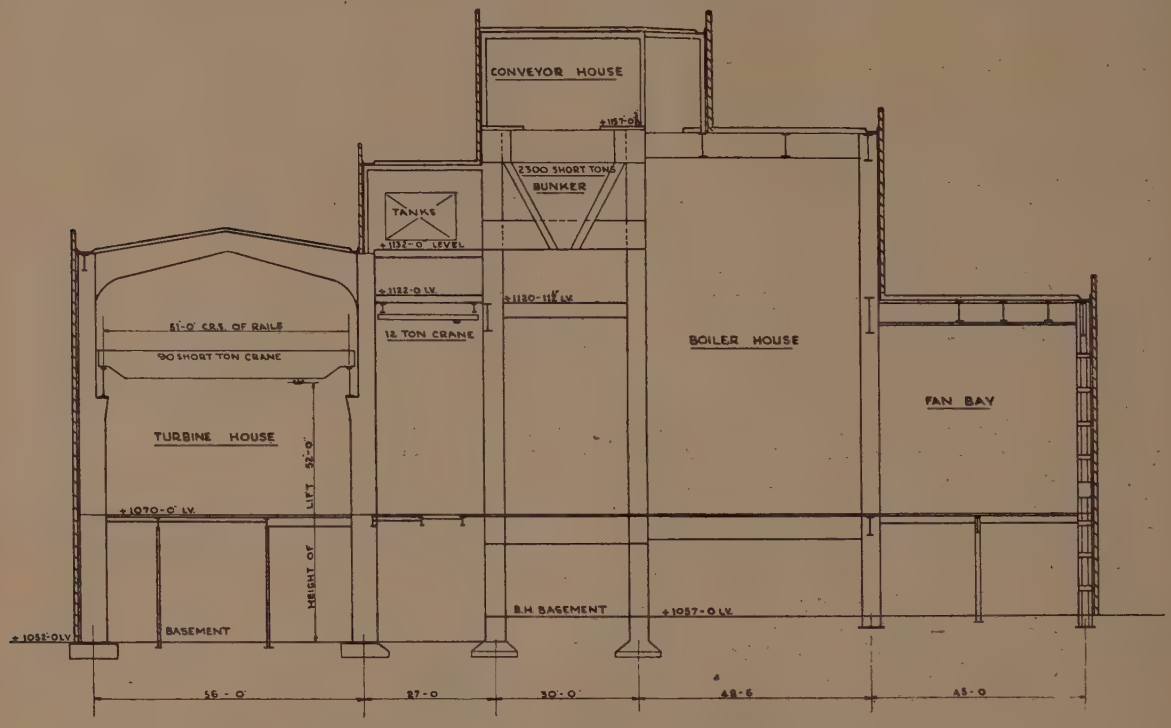

Plate 3—Umgeni Power Station. Transverse section

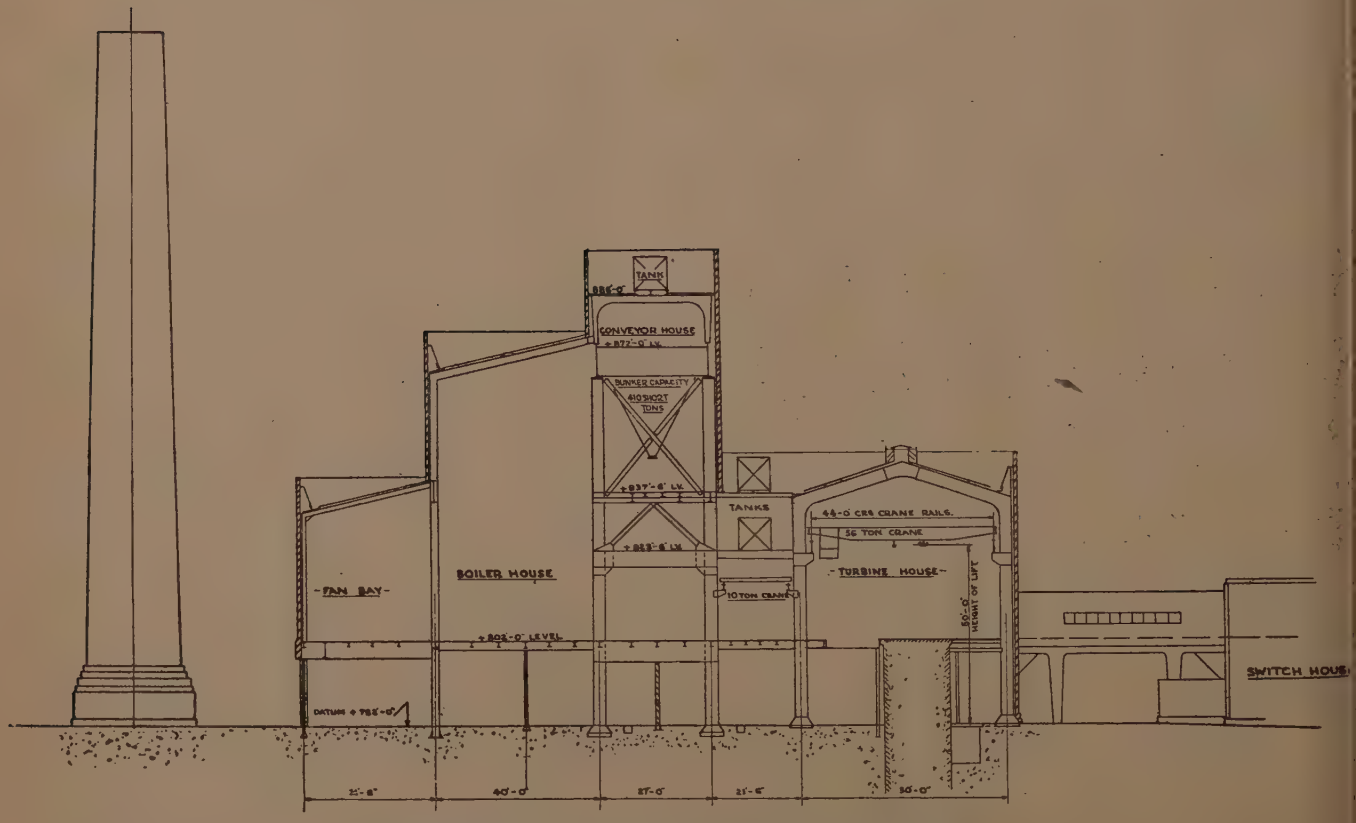

Plate 4—Hex River Power Station. Transverse section

The result of these differences is a decrease of about 5 per cent. in the amount of steel required per MW as ompared with Umgeni.

We now pass to Vierfontein where local conditions, lant layout and structural design are quite different om those of the two previous stations.

This station is sited away in the veld; the nearest owns of any consequence being Klerksdorp 22 and Croonstad 53 miles away, so that there is no serious bjection to visible sloping roofs covered with corrugated galvanised iron.

The generators are placed side by side transversely, he boilers are in a double row with the bunker aisle etween them, and an off-loading bay is provided.

In order to keep step with the plant erection programme the turbine house had to be erected before the oiler house so that it had to be self supporting in that no

side by side transversely, but being 60 MW machines instead of 30 MW the width of the turbine house is greater; the crane span is increased from 79 ft. 5¼ in. to 115 ft. 0 in. and the capacity of the crane is increased from 75 long tons to 125 long tons. The boilers are in a single row.

At Taaibos the only flat concrete roof is that over the auxiliary bay, the remainder being flat sloped, covered with protected metal deep V-beam sheeting. Main building and crane columns are at 36 ft. 0 in. centres.

From Plate No. 6 it can be seen that the design as a whole is a combination of simple braced frame and rigid portal frame construction. Here the boiler house is being erected first, the bunker aisle obviously being the back-bone of the structure as a whole.

The erection of the turbine house follows that of the boiler house, bay by bay, and so does not have to be self

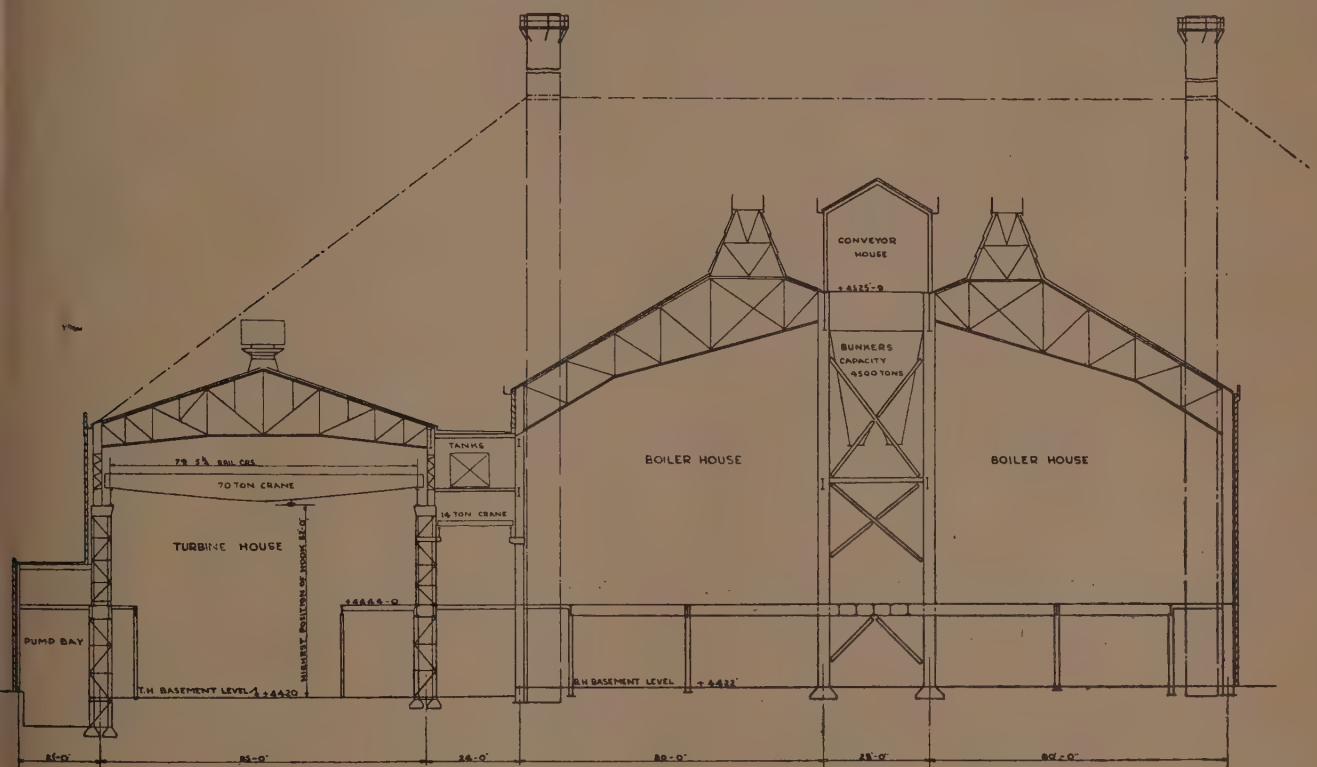

Plate 5—Vierfontein Power Station. Transverse section

horizontal loads could be transferred from the turbine ouse to the boiler house structure, until the boiler house vas erected.

The only concrete flat roofs are those over the auxiliary ay and C.W. pump bay. The remaining roofs are loping and covered with corrugated galvanised iron heets giving a dead and superimposed roof load of pproximately 20 lb. per sq. ft.

Plate No. 5 shows that this station is designed virtually as two separate simple braced frame structures. The unker columns, vertically braced in both directions, orm the back-bone of the boiler house. Main building nd crane columns are at 30 ft. 0 in. centres throughout, nd the 75 ton crane span is 79 ft. 5¼ in.

This radical change in design resulted in a decrease in he weight of steelwork per MW of 49 per cent. on that sed at Umgeni and 40 per cent. on that used at Hex River.

Plate No. 6 shows the cross section of Taaibos Power Station, which again is situated in the veld, some 2 miles from Vereeniging. Like Vierfontein an offoading bay is provided and the generators are placed

supporting, all horizontal forces being transferred to and withstood by the bunker columns.

What appears to be a very peculiar form of construction at the top of the boiler house requires some explanation. The Taaibos boilers are P.F. boilers each one evaporating 580,000 lb. of water per hour and weighing some 2,400 tons each including air heaters and economisers; by far the largest boilers ever built in South Africa.

Unlike the usual boiler supported from the floor, these are hung from a grid, which is represented in Plate 6 by the heavy beam spanning between the tops of the box columns. The Taaibos boiler house columns have, therefore, the additional duty of carrying the boilers, the load on each being increased by some 400 tons thereby, and this point must be borne in mind when studying Table 1 and comparing Taaibos with the three previous stations, at all of which the boilers are supported from the ash basement floor. This boiler supporting steelwork for Umgeni, Hex River and Vierfontein is not included in the figures given in Columns 9, 10 and 11 of Table 1. The two pin arch above the boiler grid provides the necessary

space for the withdrawal of the pendant superheater tubes.

In spite of the increased crane span and load and the very heavy duty imposed on the boiler house columns this design requires slightly less steel per MW than the Vierfontein design.

Before proceeding further it should be noted that one of the minor reasons for the greater weight of steel at Umgeni and Hex River is that at both these stations the specification called for the design to be in accordance with B.S.S. 449 of 1937, whereas Vierfontein and Taaibos were in accordance with B.S.S. 449 of 1948 as amended.

Wilge No. 1 Power Station is of peculiar interest in that the boilers are of the outdoor type and therefore no boiler house building is required. Only a turbine house is provided, in which the generators are placed end to end longitudinally. The structure is a simple braced frame

Columns 2 and 3 of Table 1 show that the arrangement of the plant at Umgeni and Hex River is the same but that the capacity of the generators at Hex River is only 2/3 of those at Umgeni, though the individual capacity of the boilers is slightly larger. This is reflected in columns 6 and 7 which show that by using a larger capacity machine, less volume per kW is required in the turbine house, and that the decrease in the volume of the boiler house at Hex River as compared with Umgeni is almost in direct proportion to the decrease in the total boiler capacity.

Column 2 shows that the arrangement of the machines at Vierfontein is the same as at Taaibos, but that the Taaibos machines are each double the capacity of those at Vierfontein. Here again Column 6 shows that the larger machine requires less building volume per kW but, Column 7 shows that the single row of boilers at Taaibos, although the individual boiler capacity is more than

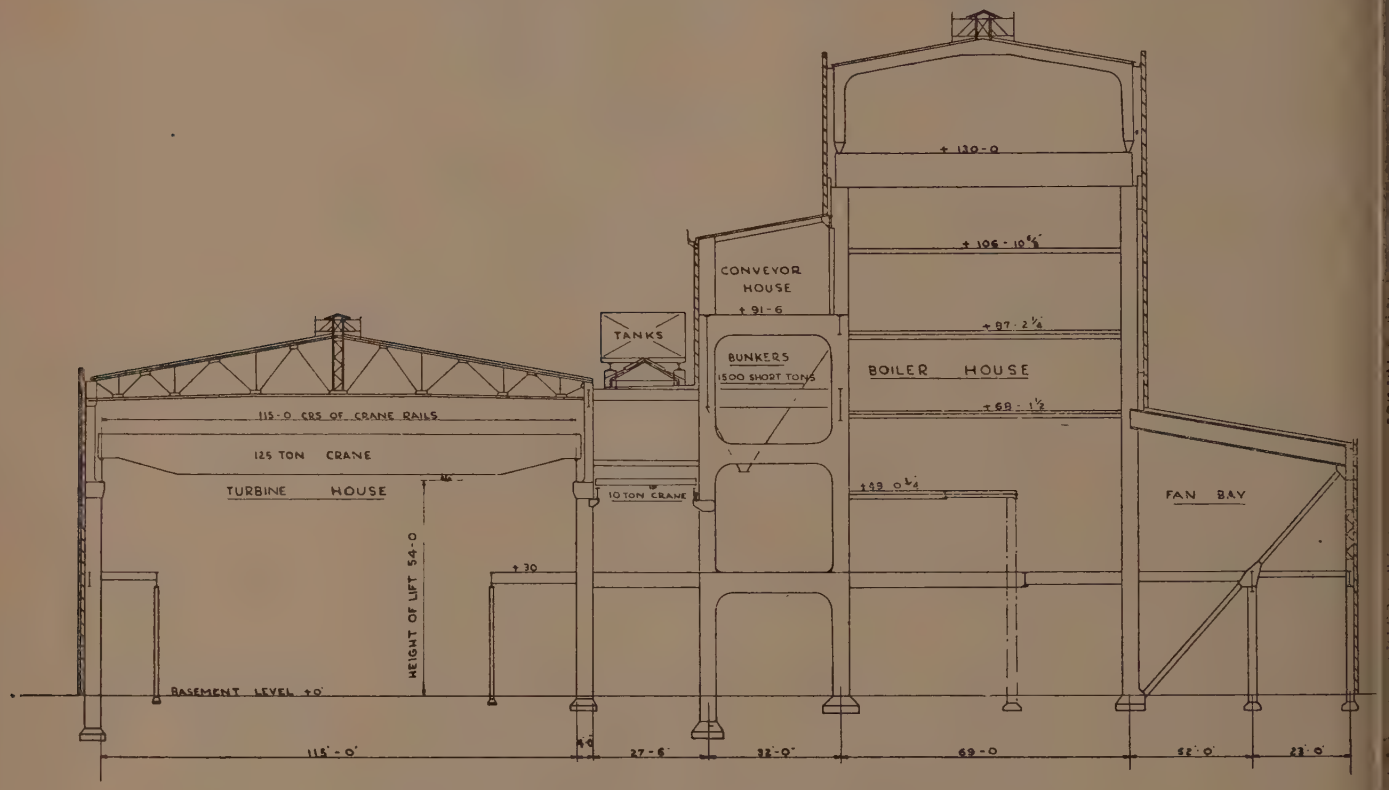

Plate 6—Taaibos Power Station. Transverse section

supporting a 75 ton crane with a 44 ft. 0 in. span, and the main building and crane columns are spaced at 30 ft. 0 in. centres.

The weight of steel used is, in the long run, decided by three factors, namely :—

(a) the type of design adopted, that is, braced frame, portal frame or a combination of these.

(b) the volume provided to house the given plant and

(c) the specified restrictions or conditions.

The designer's problem is to arrive at the most economical combination of (a) and (b) in the light of (c).

The type of design is influenced to a certain extent by the arrangement of the plant, as for instance whether there are any large dead loads such as fans and tanks to be supported on the roofs.

The volume of the buildings is controlled as to the minimum by the arrangement and the size of the plant to be housed, though, as mentioned before, it should not be the designer's aim to provide an absolute minimum of volume but a minimum consistent with the easy operation of the station.

double that of Vierfontein, requires 32 per cent. less boiler house volume. This can only be due to the different design of boiler because Umgeni and Hex River both with single rows of stoker fired boilers, have very little less boiler house volume than Vierfontein with its double row of stoker fired boilers of much the same individual capacity.

The structural engineer is not of course concerned with the choice of type of boiler to be used, i.e. stoker fired or P.F., and is only concerned with the boiler arrangement if the deciding factor is the structural economy of the boiler house.

If the latter were the case then his advice would be obvious for a boiler house accommodating a double row of boilers will require less steel than one accommodating the same boilers in a single row, since the same or nearly the same design of bunker columns is required in both cases, but the single row will require twice as many columns and twice the length of roofing over the conveyor house above the bunkers as the double row.

As a matter of interest the space required per lb. of water evaporated per hour at each of the four stations is as follows :— Umgeni 3 cu. ft., Hex River 2.53 cu. ft., Vierfontein 2.95 cu. ft. and Taaibos 2.59 cu. ft. (Pretoria "B" only 2 cu. ft.).

As is to be expected the Wilge generators being larger than those at Hex River, though arranged in the same way, require slightly less volume per kW.

Column 10, which from the point of view of the overall cost of the station is the most important of them all, represents the result of structural design controlled by size and arrangement of plant and other stipulated conditions.

It shows that the structural design of Vierfontein combined with the changed layout of the plant and the simpler conditions compared with the Hex River and Umgeni is very much more economical than either of those two stations.

It also shows that by doubling the capacity of the generators and boilers at Taaibos further economy was obtained.

The figures in Column 10 do not however show a fair overall comparison between Taaibos and the other stations because, as described above, the Taaibos boiler house structure also supports the boilers. In order to obtain a true comparison the weight of steelwork in the boiler substructures must be added to Umgeni, Hex River and Vierfontein. When this is done the figures become 73.3 tons per MW for Umgeni, 64 tons for Hex River and 43.2 tons for Vierfontein. The 28.5 tons per MW for Taaibos now shows a very marked advantage over the other three stations, which can only be due in the main to the very large increase in the individual generator and boiler capacity, and the use of top hung boilers.

From the purely structural design point of view Column 11 is the most important, giving, as it does, the weight of steel required to enclose one cubic foot of space. It shows that the Vierfontein design is by far the best from that point of view. Even when the figures are amended to include the boiler substructures, Umgeni becomes 3.02 lb. per cu. ft., Hex River 2.55 lb. and Vierfontein 1.58 lb. Vierfontein remains structurally the most economical design. It is of interest to note that of the British stations shown in Table 1 only Bankside and Cliff Quay are of the same order as Vierfontein as regards weight per cu. ft.

Column 12, in which the weight of steel in the turbine house is considered separately proves again that the larger the individual capacity of the generators the less steel per MW is required.

This column also shows again that the simple braced frame structure is more economical than the rigid portal frame for this purpose.

The tonnage of steel in the boiler house structure per 1,000 lb. of steam produced is shown for each station in Column 13, and, in brackets, the tonnage including the boiler substructures.

Here again the economical design of Vierfontein is demonstrated.

Columns 14 and 15 have been reduced to a common base of £100 per ton of steel erected and are therefore in direct proportion to Columns 11 and 10, and consequently give the same deductions.

So far all Escom's power station buildings have been constructed of steel. For purposes of comparison the figures have been given in Table 1 for Pretoria "B" Power Station, the property of the Pretoria Municipality, which is a reinforced concrete frame structure.

It will be noted that the volumetric capacity provided is much the same as in the Escom stations. It will also be noted that there is a large saving of steel per cu. ft. but that nevertheless the cost of the complete frame per cu. ft. of volume enclosed shows little saving over Vierfontein and the cost per kW installed a saving of 28 per cent. over Taaibos and Vierfontein, although the Taaibos figure includes the boiler supporting steelwork, which the Pretoria "B" figures do not.

Each type of structure, of course, has its advantages, but we feel that the saving in steel and the reduction in cost per kW is more than outweighed by the fact that, other things being equal, with the steel frame structure, the turbine house crane girders can be made available much earlier than would be the case with the concrete structure, and this at present is a matter of great importance to Escom.

As stated above there are too many variable conditions affecting the design of power station buildings to make it possible to draw any fixed conclusions from Table 1, but it does seem obvious that the fully rigid portal design used for both Umgeni and Hex River will not generally lead to the most economical structure. On the other hand it would appear that the Vierfontein and Taaibos design in which the bunker aisle is designed as a rigid structure forming a backbone to the remainder of the building, which is then designed as a simple structure around it, will generally require the least amount of steel for the greatest volumetric content, and the lowest cost per kW installed. It should be noted too, that the Umgeni type of structure must of necessity cost more per ton owing to the difficulty in erecting, lining up, plumbing and grouting in without the help of bracing.

There remains the old question, "Should the generators be placed longitudinally or transversely in the turbine house ?" We are satisfied that, other things being equal, the answer is given by a comparison of Wilge turbine house with that at Vierfontein.

The capacity of the individual machines is the same at both stations but at Wilge they are placed longitudinally while at Vierfontein they are placed transversely.

With the exception of the crane span the design of the two turbine houses is the same.

It seems fairly obvious that the increase in weight of the crane columns and roof trusses due to the increased span for the transversely placed machine will be more than offset by the increase in weight of steel due to the larger number of bays required for the longitudinally placed machine with the consequently smaller crane and roof truss span. Both buildings are designed to take the full wind load on one side.

The actual figures are given in Column 12, which shows that at Vierfontein a 30 MW machine placed transversely requires 9.1 tons of steel per MW, whereas at Wilge the same machine placed longitudinally requires 11.3 tons of steel per MW to house it.

In other words, by placing a 30 MW machine transversely instead of longitudinally there is in this case a saving of 66 tons of steel per machine.

In addition, when considering the overall cost, the cost of the brick or other cladding must be included and the cladding at Wilge, excluding the gable ends will be 1½ times as much as at Vierfontein, because the longitudinal machine occupies three 30 ft. 0 in. bays, whereas the transverse machine only occupies two 30 ft. 0 in. bays.

The proof of the pudding is in the eating.

To leave the purely structural aspect of this question for a moment, in South Africa, we consider that a station with longitudinal machines is not so easily operated as

one in which the machines are placed transversely, owing to the increased length of the turbine house. Take for example Klip Power Station where there are 12 33,000 kW sets and four 7,000 kW house sets all placed transversely, and the turbine house is in consequence some 820 ft. long. If these same machines had been placed longitudinally the length of the turbine house would have been something like a fifth-of-a-mile; a three minute walk not appreciated by any shift engineer.

There is one factor affecting the building design which is apt to be brushed aside as of little importance, but which, if not given careful consideration may add considerably to the weight of steelwork, and that is wind pressure. In South Africa we are not greatly troubled with high velocity winds. The highest instantaneous gust recorded was 95 miles per hour, but the highest average wind velocity for a single hour was only 55 miles per hour, which, according to the B.S. 449 of 1949, represents a pressure of only 12.7 lb. per sq. ft. at a height of 90 ft., whereas in the past many South African designers have made a habit of allowing for a pressure of 30 lb. per sq. ft. over the whole exposed area, regardless of available meteorological data, thus more than doubling the bending moments due to wind, quite unnecessarily.

As a result of the comparison of facts, which I have in this paper endeavoured to put before you, and of the deductions which I think have been fairly made, we have come to the conclusion that in order to achieve the most economical power station building from the point of view of the overall cost of the station we must :—

(a) avoid the use of rigid portal frame construction for the whole building, unless, perhaps, the use of the plastic theory and collapse method of design will safely produce a considerable reduction in weight.

(b) not preclude the use of vertical bracing.

(c) avoid as far as possible the use of heavy roof covering.

(d) avoid if possible the placing of heavy plant on flat roofs or at high levels.

(e) carefully consider wind velocity to be expected at the site.

(f) preferably place the generators transversely.

(g) use plant of the highest possible individual capacity and lastly,

(h) within the limits of the plant requirements and site conditions allow as much latitude as possible to the structural designer in the specification accompanying the original enquiry for tenders for the building structure.

Regarding the last item my personal opinion is that the most economical design is obtained by calling for tenders for the design, fabrication and erection of the required structure from a number of reputable structural steelwork contractors, so that full rein can be given, in competition, to the individual designer's ingenuity and experience limited only by the recommendations of B.S. 449, and the approval of the client.

This is Escom's practice and our specification normally reads :— "subject to these requirements, he (the Tenderer) is to design the work in the most advantageous manner, having in mind economy in weight and erection and general suitability for the plant to be installed in the buildings. Rolled steel sections or welded built up sections or both may be used having in mind the availability of steel shapes and plates."

Apart from this the only initial limits we impose are wind loading, superimposed floor loads, crane spans and loads, bearing pressure, and description of the cladding we intend to use.

Some of the "Musts" listed above are perhaps self evident, but the justification for their inclusion here is that in this case they are not merely opinions but are statements backed, it is hoped you will agree, by facts.

In conclusion, I have to thank the Management of Escom for permission to read this paper.

Discussion

THE CHAIRMAN proposed with very great pleasure a vote of thanks to Mr. Frost for his quite remarkable review of the structural work in power stations in South Africa.

He was particularly pleased to welcome Mr. Frost because the meeting was one of the very rare occasions on which the members of the Institution had had the privilege and pleasure of listening to a paper by a member of the South African Branch; and having heard the paper, he was more glad than ever to propose a very hearty vote of thanks to Mr. Frost.

Mr. P. PERCIVAL, after expressing his great interest in the paper, said it was a very difficult matter, as Mr. Frost would appreciate, to compare stations on the basis of tonnage or cu. ft. per megawatt, as so many variables came into the comparison. There was a great variety of ways of treating a building. Sometimes a station had to be erected within a restricted space, which meant that plant had to be raised into the air, and we obtained entirely different results on that account.

In his view a much better basis of comparison was tonnage per cu. ft., provided one was comparing like with like.

Speaking as a mechanical engineer, Mr. Percival agreed that the civil engineer and the structural engineer often had a very poor run at the job; but, when all was said and done, the actual structure of the power station represented perhaps 7½ per cent. of the total cost of the job; that was a guess, but at any rate it was less than 10 per cent. It was obvious that the structural man had rather to take a back seat, because the plant which was accommodated in the structure was extremely complicated and costly and, of course, costs were increasing all the time.

The Bankside station in London was included by Mr. Frost in his table of comparisons. One of the great loading-up factors in a power station structure was the coal bunker, which might be designed for 600 tons per boiler or more and we should not compare that station with the other stations listed as it was oil fired and therefore had no bunkers.

Mr. FROST said he was not trying to make any comparison between the South African and the British stations, for he knew nothing of the latter. He had included the information relating to them purely as a matter of interest.

Mr. B. A. E. HILEY (Member of Council), referred to the comparisons given by Mr. Frost of volume per kW and cost per kW of installed plant. At a glance it would appear that in South Africa they were doing very well to say the least, but on closer examination we must be careful before drawing conclusions.

In South Africa they were not called upon to include electrostatic precipitators, gas cleaning plant, etc., as out in the veld they could discharge the chimney emissions without let or hindrance. Further, in this country we had loading bays at either end of our stations to accommodate the rail and road deliveries of heavy plant units. In the Union all was delivered by rail.

From the figures given it would be seen in Column 5, that volume of buildings per kW of installed plant was less than in this country. Mr. Hiley suggested comparing Pretoria, on which his firm had a hand, with Hams Hall " B " Station. Both were reinforced concrete framed structures.

At Pretoria the building volume was 47.5 cu. ft. per kW of plant installed whereas at Hams Hall " B " 68.7. One was inclined to ask why. The reason was that at Hams Hall " B " which compared favourably with other British stations, they included extensive offices, workshops, and all amenities usually built outside the station, as well as two extensive loading bays and one extensive bay, some 900 ft. by 40 ft. to incorporate the electrostatic precipitators; also brick or concrete chimneys housed in station buildings in lieu of steel ones with guy ropes to steady them. If we cut all these out, we would find that the volume of buildings per kW of plant was comparable with that of Pretoria.

On the matter of cost per kW the following might be interesting :—

1939/45 (War period)

Hams Hall " B "—1st half £3.8 per kW

Hams Hall " B "—2nd half £5.0 per kW

If we allowed £1 per kW for the additional work as above described these figures of cost were reduced to £2.8 and £4 per kW respectively.

He compared these figures with Hex River (1950) at £4.36 per kW and Pretoria (Ferro-concrete frame) £2.12 per kW.

The other station on Mr. Frost's list of " British Stations," was Brunswick Wharf which his (Mr. Hiley's) firm had designed. This station was built in the bottom of East India Dock and on a most difficult site, as those who knew it would agree. No comparison was possible here with any of the stations in the Union.

He felt that the tables given should be related to dates of construction, since values from 1939 in Britain varied from 3 to 3½ times the 1939 figures, and on an ever increasing cost curve.

Finally, he thought comparisons should be made only by remembering the following :—

(1) The mechanical electrical layout requirements to which Mr. Frost referred.

(2) All calculated money values on Mr. Frost's schedules should be attuned to the dates on which stations were built and relative costs per kW inserted.

(3) We in Britain provided generally, more space and included two loading bays for use by rail or road. Electrofilters were incorporated here, and were not included in the Union. Chimneys were generally incorporated in main buildings and also flues and fan gear were housed as at Hams Hall " B ". The Administration and Amenity Blocks and workshops were usually included in the British figure.

(4) Sites were difficult in Britain, whereas Africa was large and rarely called for piling or the use of indifferent ground. One only had to consider the Union sites with any London station to appreciate this point.

(5) In Britain plans had to be submitted to the Royal Fine Arts Commission which studied architectural treatment of the whole.

(6) Since 1939 contractors had had to cover in their tenders for shortages of materials and labour and comply with innumerable Acts and restrictions.

Mr. Hiley thanked the author for a splendid paper.

Mr. Frost said that since he had been round several stations in this country he had realised that they included quite a number of things, such as workshops, as part of the main structure, which had never so far been done in South Africa.

Mr. Frost again pointed out that he had had no intention of comparing the South African and British stations on a cost basis, he also wished to correct Mr. Hiley's assumption that South African power stations had no off-loading bays. They did, in fact, always have one, sometimes two.

Mr. Frost also stated that the calculated money values in his schedule were strictly comparable as they had all been reduced to a common base of £100 per ton of steel erected.

Mr. Leslie Turner (Past-President) referred to his Presidential visit, with the Secretary, to the Union of South Africa Branch, which had kindly arranged their tour to most of the major works in progress in the Union. Around Johannesburg these had naturally included visits to power stations completed and under construction. They were most impressive installations, the Klip Station being the largest in the Southern Hemisphere. Although some were approaching or had surpassed their originally planned size there were considerable structural works proceeding, in one case a large underground tunnel to keep pace with new ideas and to increase efficiency, while in another the station was in process of extension incorporating some of the world's largest concrete cooling towers. This was visual proof of revised or second thoughts on the part of electrical engineers and of the unpredictable growth in demand for electrical energy, which the lecturer had emphasised.

It was to Mr. Frost that he and the Secretary were indebted for the visits to these imposing constructions, in addition to his most generous hospitality and time given in other directions, particularly as Vice-Chairman of the Branch and later as Chairman, to which office he was elected at their Annual General Meeting, which he and the Secretary had the privilege of attending.

It was, therefore, particularly gratifying to Mr. Turner to add his welcome and thanks to Mr. Frost in his capacity as lecturer at Headquarters that evening, and a real pleasure to have listened to his detailed exposition of the structural work and his general theme of Power Station Development in South Africa.

He had enjoyed every minute of the lecture and the well-informed discussion, and he appreciated, especially, the value of the comparative figures Mr. Frost had compiled relating to the structural economics of power stations.

Mr. P. L. Capper, T.D., M.Sc.(Eng.), A.M.I.C.E. (Member of Council), expressed particular interest in Mr. Frost's remarks concerning the electrical engineers' lack of appreciation of construction problems, and said he would be interested to know whether in Johannesburg the students of electrical engineering had to do a certain amount on structures. That was the case in most universities here, so that electrical engineers should go forth with at least an elementary knowledge of structural engineering. The electrical people, on the other hand, had their own back on the civil engineers by insisting on their learning a considerable amount of electrical engineering ; in addition to a useful knowledge of the terms used in electrical engineering, such as megawatts and micro-henries, and so on, they must also learn details of the windings of electric generators, numbers of poles and things of that sort, which were perhaps of less interest to the structural engineer.

Mr. Capper added his thanks to Mr. Frost for his interesting paper.

Mr. FROST replied that he did not know whether the students of electrical engineering in Johannesburg did any structural engineering training, but he imagined they did ; in fact, many of them had been trained in this country. But he believed the knowledge the younger ones had of structural engineering was purely from text-books and was not backed up by any practical experience of the subject.

Mr. D. N. MITCHELL (Member) said he had experienced the difficulties that had been mentioned, in the construction of industrial undertakings, such as steelworks, and he assured the meeting that the situation was similar in those cases as with power stations, but he suggested that there were quite sound explanations.

When dealing with a large industrial undertaking, it should be remembered that the industry did not have an opportunity to put up a new structure of different and more modern type very often, and that research and development were going on all the time. Speaking of a recent contract he said the work of building had taken about four years ; before that it had taken several years to plan and design. During the four years of construction the detailing was going on ; and research was going on also. Part way through such a contract a client who wanted the most modern plant possible would learn of the introduction of a new type of machine, and decide that this should be incorporated into his plant because he might not get another opportunity for many years.

If the electrical and mechanical engineer, and also the client would agree that they were going to have (say) a 1956 plant, but not complete this until 1958, then there would be much less alteration.

Mr. FROST agreed with everything Mr. Mitchell had said. He added that research was proceeding also in civil and structural engineering, but he doubted very much whether the client or engineer would change his ideas in the middle of a job, even if something were then introduced which they had not seen before, unless it could be easily used without great extra cost, or unless it actually saved money.

Mr. R. G. STEFANELLI, as an electrical engineer, said he was very pleased to hear that somebody had come to the defence of electrical engineers. The real answer, he suggested, was that the detailed layout of plant was, in the nature of things, a little behind the civil engineering construction, because the civil engineering works had to go in first. Often when the detailed plant layout was settled, the originally proposed position of, say, a cable tunnel, could with advantage be moved.

It was a little surprising that ESCOM apparently were not able to decide on the ultimate capacity of their stations reasonably in advance. One would have thought their knowledge of the annual rate of load growth in the area to be supplied would permit this within certain defined limits ; and with this in view, it would be possible to lay out the station to enable due regard to be paid to railway facilities and the location of office blocks, chimneys, and so on. He was a little surprised that ESCOM found it rather difficult to arrive at such decisions.

Mr. FROST assured Mr. Stefanelli that it was really difficult to make up one's mind about what was to be built. Particularly since the last war a number of new gold mines had come into being in the Free State, and they came along so rapidly that ESCOM had had great difficulty in keeping pace with their requirements.

Mr. H. D. WATSON, who gave some figures of costs, said the cost of coal of 8500 B.Th.U./lb. into bunkers when the Klip station was started up, was 2s. 6d. per ton. At the Wankie Colliery, about 70 miles south of Victoria Falls, the pithead price was 13s. 6d., the calorific value of the coal being about 13,500 B.Th.U. per lb.

The cost of the Klip station was round about £16 5s. per kW in 1936. Its capacity was about 400,000-440,000 kW. The present-day costs of power stations in this country were £50-£60 per kW, which compared favourably after making allowances for increase of costs since 1936.

Mr. Watson drew attention particularly to the Brimsdown station building. Mr. Frost, he said, had quite rightly mentioned that the tonnage of steel in power stations varied considerably with the location, the arrangement and the æsthetic values that were demanded. But the station now referred to had brickwork to a height of about 6 ft., so that it would withstand shock from lorries; above that height the covering was made of asbestos-cement troughed sheeting about $\frac{3}{8}$-in. thick, clipped onto the building framing, and it had been in position for 14 years. A great advantage of using that type of covering was that if the station were widened or lengthened the sheeting could be taken down and used again. He did not suggest this form of covering was applicable to South Africa, as the climatic conditions there were frequently severe ; he mentioned, for example, that sometimes there were hailstones as big as eggs and small oranges.

Another form of covering used in South Africa was corrugated iron. It was rather doubtful, he said, that the British Electricity Authority would accept its use in power stations in this country.

Finally, he respectfully suggested that it was well worth while to read and compare the reports on electricity supplies of the British Productivity Team which had visited America and the American Productivity Team which had visited stations in Britain, for not only did they both compare the power stations building volumes in the two countries, but they also reviewed the general position very comprehensively.

Mr. FROST, confirming what Mr. Watson had said about the particular station to which he had drawn attention, said it was undoubtedly a very good-looking structure. Much as we should like to make much wider use of the asbestos-cement material, that was quite out of the question for buildings housing electrical plant in South Africa. The heat of the sun was such that the material became very brittle after a year or two, so that if dismantling were undertaken for extension or other purposes the percentage of breakage was usually very high. There had been cases of men walking over a roof of that material and falling through it, due to its brittle condition. There was also the point that the hailstones in South Africa were quite large, they were quite likely to damage badly the roofs of motor cars, and could certainly smash all the glass in a large window, so that the effect on cement asbestos sheets could be well imagined. One could not possibly afford to risk that happening in a structure which housed, say, twelve 33,000 kW generators, and the consequent possible entry of a deluge of rain.

Mr. J. W. MEREDITH (Member), who had spent five years in South Africa and had met Mr. Frost there on several occasions both on business and socially, was glad of the opportunity to join in welcoming him to this country.

Discussing the problem of site organisation when nstructing large works, he said he had had some perience of industrial plants such as boilers, coking ants, etc., which had to be built miles from anywhere, d it was quite an achievement to organise the necessary bour. For example, on one occasion he was concerned ith the construction of a coke oven plant of the value of ¾ millions, and it was necessary to house and maintain o Europeans and 400 natives on the site. It was portant, therefore, that Mr. Frost had mentioned that one of the problems to be tackled by the engineer.

Mr. Meredith added his appreciation of the opportunity of hearing Mr. Frost present so excellent a paper.

Mr. FROST agreed that the housing and maintenance the men engaged in the construction of a large plant as really a major problem. In South Africa the rocedure was perhaps different from that adopted in his country. If, for instance, a 300,000 kW station ad to be built in the veld, the operating staff must be oused there. That meant something like 120 houses narried quarters) and quarters for about 40 single men nd 300 natives. But that was not nearly sufficient r the contractors' people while on the job. Obviously, owever, it was not economic to build large numbers of ouses which would not later be required.

The CHAIRMAN commented on the absence of bracing t the Umgeni station, for it seemed to him to be a tructure in which bracing would be economically desirble as there were very considerable lateral forces to be esisted. There was a very heavy crane in the turbine ouse, which would introduce large lateral loads ; also he building was necessarily high, and therefore was able to very considerable wind forces. Therefore, he ould have thought it reasonable for the design to include racing to provide the necessary lateral stability, and in act he understood that this was done in all cases referred o in the paper except Umgeni.

He asked whether in the case of Umgeni it had been tipulated in the general specification that the frames ust be of the portal type without bracing, or whether he matter was left to the individual designers. If it ere stipulated that bracing must not be introduced, he ondered if there were any special reason, because therwise he would have thought that the designers ight have been given freedom to investigate the elative merits and economics of a rigid framed conruction without bracing and the alternative of a entral " spine," properly braced, which would provide he necessary lateral stability to the whole building. If his had been done, he was convinced that the " spine " esign would have shown considerable economic advange over the rigid frame design.

Mr. FROST replied that it had been stipulated in the eneral specification of the building that there should e no vertical bracing. The structural designer had herefore had no choice in the matter.

Mr. Frost said that he believed the decision to omit ertical bracing was influenced by a desire to avoid any bstruction to high pressure pipework between the oiler house and turbine house, and to give a clear view own the firing aisle, but he pointed out that longitudinal racing would only be required in one bay in any case, nd the transverse vertical bracing could be designed to rovide a minimum of obstruction in the firing aisle.

Mr. Frost agreed with the Chairman's remarks on the entral " spine " type of construction and said that he lt that the figures given in Table I proved that it was he most economical design.

Mr. SCOTT WHITE expressed his great pleasure in again meeting Mr. Frost and in listening to his most interesting talk. He had visited South Africa twice last year and had returned immensely impressed with the potential developments in the country and the high state of efficiency already achieved. Mr. Frost had ventured to remark that many engineers in England were probably unaware of the advance in design in South Africa, which might equal the best in this country. His talk had clearly shown where South Africa stands.

Mr. Frost had indicated the difficulty of judging how far provision should be made for future supply of power. This could readily be appreciated if one reflected on the almost incredible growth of the Union during the last 70 or 80 years. Johannesburg had developed during that time from a small mining town to one of the finest cities in the world—quite comparable with any counterpart in America. In the area around Vereeniging, industry was developing at an amazing rate and it was difficult to estimate what the future demands on power supply would be.

The time available for discussion having expired, written contributions were invited.

The CHAIRMAN expressed very grateful thanks to Mr. Frost for the lucid, comprehensive and easy way in which he had replied to the discussion, matched only by the way in which he had presented his paper. He had provided an extremely interesting and pleasant evening, and the meeting was really grateful to him.

(The meeting then closed).

Written Discussion

Mr. F. T. WALTON (Graduate) writes : Mr. Frost has stressed the importance of joint consultation between the several engineering services during the construction of a power station in order that serious misunderstandings and delays may be avoided when a sudden need for structural changes due to (say) electrical modifications are introduced.

I would like to enquire what form this consultation does take (or in Mr. Frost's view, should take). Are formal meetings with all branches represented held, or does the element of casualness which typifies so many British sites occur ?

We as Structural Engineers appreciate that the client is " doing the paying " and that we are the " servants " and not the " masters," but if we are to be given a chance to produce our best designs and suggestions when last-minute changes are required, then some form of ordered consultation at which we can be fully represented is clearly necessary.

Mr. FROST replied : Mr. Walton's point regarding co-operation between the various branches of engineering concerned in the building of a power station is very pertinent and one which has not been lost sight of in South Africa. When it has finally been decided to build a new station we call a meeting of all the engineers concerned to discuss the layout, plant positioning, etc. These meetings are repeated until everyone is satisfied. During the design and construction periods similar meetings are held as required, to clear up any problems which invariably arise.

Notice of any change of plan, design or plant by any engineer is immediately passed on to all the other branches in case they may be concerned. There are slips, of course, but on the whole this plan works extremely well. Such planning is particularly useful to the structural engineer, for without it he cannot even begin to produce his construction " time " chart.

Institution Notices and Proceedings

PRESIDENTIAL ADDRESS—SESSION 1953-54

A General Meeting of the Institution of Structural Engineers will be held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 1st, 1953, at 6 p.m., when Lt.-Colonel R. F. Galbraith, M.C., B.Sc., M.I.Struct.E., A.M.I.C.E., will be installed as President for the Session 1953-1954, and will give the Presidential Address.

FORTHCOMING MEETINGS

Thursday, October 22nd, 1953

Ordinary General Meeting for the election of members at 5.55 p.m., followed by a Joint Meeting at 6 p.m., with the British Iron and Steel Research Association and the Engineers' Group of the Iron and Steel Institute, when Mr. J. S. Terrington, B.Sc.(Eng.), A.M.I.C.E., A.C.G.I., A.M.I.Struct.E., and Dr. J. M. Hawkes, B.Sc., A.M.I.C.E., A.M.I.Struct.E., will give a paper on "Crane Gantry Girders for Steelworks."

Thursday, November 12th, 1953

Joint Meeting with the British Section of the Société des Ingenieurs Civils de France at 6 p.m., when Monsieur L. P. Brice will give a paper on "Different Types of Wharves and Maritime Structures recently executed in France."

Thursday, November 26th, 1953

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E. (Member of Council), and Mr. K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E., will give a paper on "The Reconstruction of a Soaking Pit Building."

Thursday December 17th, 1953

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. W. Hamilton, A.M.I.C.E., and Mr. G. P. Manning, M.Eng., M.I.C.E., will give a paper on "Some Structural Uses of Aluminium Alloy with special reference to Domes."

Members wishing to bring guests to the Ordinary and Joint Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and overseas on January 5th and 6th (Graduateship) and January 7th and 8th, 1954 (Associate-Membership).

CONFERENCE ON WELDED STRUCTURES

A Conference is to be held in London on November 23rd-27th, 1953, to review present knowledge and outstanding problems in the design, fabrication and erection of welded steel structures. Details of the programme were published in the September issue. Information regarding the Conference may be obtained from the Secretary of the Institution.

MACLACHLAN LECTURE COMPETITION

The closing date for the receipt of entries for the MacLachlan Lecture Competition is Wednesday, March 31st, 1954. Particulars of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture, and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

PARTICULARS OF THE COMPETITION FOR 1954

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1954.

2. The subject of the Lecture shall be on any aspect of structural engineering.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Wednesday, March 31st, 1954.

PAPERS FOR PUBLICATION

The Literature Committee would be glad to consider offers of papers for presentation at the Institution or for publication in the Journal.

The following is a summary of the Committee's requirements relating to articles and papers : a copy of the full conditions may be obtained from the Secretary.

(1) Articles must be of an appropriate character having a bearing upon structural engineering or upon some kindred scientific or constructional subject, and must be approved by the Literature Committee. A short title is an advantage.

(2) Contributions must be original either in subject matter or in presentation. Articles which have already been published or have been read to other organised bodies, or are carelessly prepared, will not be accepted for publication.

(3) The style of writing will necessarily vary with the individual, but authors are requested to write as plainly

d simply as their subject will allow. Papers should written in the third person.

(4) Where the subject allows, a brief introduction or nopsis should state clearly the purpose and scope of e paper or article, and the author's conclusions or commendations should be summarised at the end of e paper.

In order to facilitate the indexing of articles for erence, the author will be required in addition to epare a short precis not exceeding 25 words for insion under the title of the paper on the contents page the Journal.

(5) Illustrations are desirable where they assist in plaining the context or are fundamental to the subject. ney should not be used if unnecessary for these rposes. Illustrations may be either line drawings photographs.

(6) Line drawings must be specially prepared for production on smooth white paper or clear tracing per, with heavy main lines and large clear lettering awn in Indian ink with a mapping pen. Alternatively, e author may submit drawings on one sheet of paper ith the relevant lettering on a cover sheet of tracing per.

The printed page of THE STRUCTURAL ENGINEER is 7 in. ide by 10 in. deep. The drawings, where practicable, ould be prepared not larger than twice this size ith a view to half-scale reproduction. Unavoidably rge drawings which require reduction to one-third size less, must be specially heavy with proportionately rge lettering for clear reproduction. Ordinary working awings are not satisfactory.

(7) Where photographs are submitted they should e printed *black on glossy paper*.

(8) MS. typewritten in double spacing should be bmitted in duplicate.

Brevity is an advantage and papers should not rmally exceed 7,500 words in length.

ONDON GRADUATES' AND STUDENTS' SECTION

The following meetings will be held at 11, Upper elgrave Street, London, S.W.1, at 6 p.m. :—

Tuesday, November 10th, 1953

Mr. J. F. S. Pryke, B.A.(Hons.) (Graduate) on Underpinning."

Tuesday, December 8th, 1953

Mr. S. Turner, on "The Foundation of an 8,000,000 ft. Gasholder."

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Graduate), Bushcroft, Slipe Lane, Wormley, Herts.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Wednesday, October 7th, 1953

Joint Meeting with the Institute of Welding, Manchester and District Branch, at the College of Technology, Manchester, 7.15 p.m. Mr. S. M. Reisser, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., on "Drawing Office Procedure for Welded Designs."

Thursday, October 29th, 1953

At the University of Manchester, 6.30 p.m. Chairman's Address by Professor J. A. L. Matheson, M.B.E., D.Sc., Ph.D., M.C.E., M.I.C.E., M.I.Struct.E., followed by films, entitled, "750 Feet Television Mast at Sutton Coldfield," and "Lali Bridge." The President and the Secretary of the Institution will attend the meeting.

Thursday, November 26th, 1953

At the College of Technology, Manchester, 6.30 p.m. Three short papers by younger members of the Branch.

Friday, December 18th, 1953

Annual Dinner and Dance, at the Grand Hotel, Manchester.

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Saturday, October 10th, 1953

Annual Dinner, at the Botanical Gardens, Birmingham.

Friday, October 23rd, 1953

At the James Watt Memorial Institute, Birmingham, 6 p.m. Chairman's Address by Mr. G. E. Marsden, M.I.Struct.E.

The President and the Secretary of the Institution will be present on both the above occasions.

Tuesday, November 10th, 1953

At the Supper Room, The King's Hall (Queen Street Baths), Queen Street, Derby, 7 p.m. Mr. E. McMinn, on "Tubular Structures."

Friday, November 27th, 1953

Joint Meeting with the Birmingham and Five Counties Architectural Association. Mr. F. J. Samuely, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., on "Design and Application of Space Frames and Skin Structures."

At the James Watt Memorial Institute, Birmingham.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Friday, October 30th, 1953

At the Birmingham Civic Centre, 6.30 p.m. "The Design and Erection of B.B.C. Television Masts," by a member of the staff of British Insulated Callenders Construction Co., Ltd.

Monday, November 30th, 1953

At the James Watt Memorial Institute, Birmingham, 7 p.m. Mr. V. H Lawton, M.I.Struct.E., on "Plastic Design."

Hon. Secretary : H. L. Bramwell (Graduate), 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Wednesday, October 7th, 1953

At Newcastle. Chairman's Address, by Mr. T. Bryce M.I.C.E., M.I.Struct.E. Visit of the President and the Secretary of the Institution.

Tuesday, October 13th, 1953

The Chairman will repeat his Address at Middlesbrough.

Tuesday, November 3rd, 1953

At Middlesbrough. Mr. L. R. Creasy, B.Sc., M.I.C.E., on "The Use of Structural Steel in Buildings."

Wednesday, November 4th, 1953

The above meeting will be repeated at Newcastle.

Tuesday, December 1st, 1953

At Middlesbrough. Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E. (Member of Council) and Mr. K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E., on "Reconstruction of a Soaking Pit Building."

Wednesday, December 2nd, 1953

The above meeting will be repeated at Newcastle.

All meetings commence at 6.30 p.m. the Middlesbrough meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and those at Newcastle in the Neville Hall, near the Central Station.

Hon. Secretary : Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, October 6th, 1953

Chairman's Address, by Mr. J. C. Malcomson, B.Sc., M.I.Struct.E.

Tuesday, November 3rd, 1953

Joint Meeting with the Northern Ireland Association of the Institution of Civil Engineers. Mr. Donovan H. Lee, B.Sc., M.I.C.E., M.I.Mech.E., M.Am.Soc.C.E. (Member of Council), on "The Advantages and Disadvantages of Structural Steelwork, Reinforced Concrete and Prestressed Concrete." 6.30 p.m.

Tuesday, December 1st, 1953

Paper by Mr. R. J. N. Sweetnam, M.A., M.A.I., A.M.I.C.E. (Graduate).

Unless otherwise stated, meetings will be held at the College of Technology, Belfast, at 6.45 p.m., preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The opening meeting of the Session will be held on Monday, October 26th, when the Chairman's Address will be given by Mr. R. H. Sharpe, M.I.Struct.E. The President and Secretary of the Institution will attend the meeting.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The opening meeting of the Session will be held at Plymouth on Saturday, November 7th, and will be attended by the President and the Secretary of the Institution.

Hon. Secretary : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, October 28th, 1953

The Chairman's Address, entitled "Research Work in Soil Mechanics at the University College of Swansea," will be given by Dr. A. A. Fordham, M.I.Struct.E., A.M.I.C.E., at Cardiff.

Wednesday, November 4th, 1953

The Chairman's Address will be repeated at Swansea, and will be followed by discussion.

Saturday, November 7th, 1953

The Chairman's Address will be repeated at Colwyn Bay, and will be followed by discussion.

Monday, November 30th, 1953

Joint Meeting with the Institute of Welding, at Cardiff. Mr. S. M. Reisser, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., on "Recent Developments in Welded Structures."

Tuesday, December 8th, 1953

A meeting will be held at Swansea, when films will be shown.

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel, at 6.30 p.m.

Meetings at Colwyn Bay will be held at the County Buildings, at 6 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, October 2nd, 1953

Chairman's Address, by Mr. E. N. Underwood, B.Sc.(Eng.), M.I.C.E., M.I.Struct.E., Visit of the President and the Secretary of the Institution.

Friday, November 6th, 1953

Combined meeting with the Institution of Civil Engineers. Professor A. G. Pugsley, O.B.E., D.Sc., F.R.S., M.I.C.E., F.R.Ae.S. (Vice-President), on "Suspension Bridge Development, illustrated by some Bristol Avon Bridges."

Friday, November 27th, 1953

Mr. L. Scott White, O.B.E., M.I.C.E. (Past-President), on "Moving King Henry VIII's Wine Cellar, Whitehall."

Friday, December 4th, 1953

Combined Dance at the Royal Hotel, Bristol.

Unless otherwise stated, all meetings will be held in the University of Bristol Geology Lecture Theatre, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, October 14th, 1953

Chairman's Address, by Mr. John Dosser, M.I.C.E., M.I.Struct.E., A.M.I.Mech.E. Visit of the President and the Secretary of the Institution.

Wednesday, November 18th, 1953

Mr. A. J. Harris, B.Sc., A.M.I.C.E., on "Prestressed Concrete Beams at London Airport."

Wednesday, December 16th, 1953

Mr. W. N. Espie, A.M.I.Struct.E., on "The Handling of Bulk Cement."

The above meetings will be held at the Great Northern Hotel, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone : 34-1111. Ext. 257.

Natal Section, Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary : R. Stubbs, M.I.Struct.E., P.O. Box 2692, Cape Town.

The Reconstruction of a Soaking Pit Building*

r H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., M.I.Struct.E. (Member of Council)
and K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E.

Synopsis

The paper describes the problems encountered and ir solutions during the design and construction of a ıvy welded steel framed mill building in a highly ıgested area at the Stocksbridge steelworks of Messrs. nuel Fox & Co., Ltd.

The structure comprises a 70 ft. span crane gantry) ft. long for two vertical ingot chargers, roofed over d having a central high level roof structure housing vicing cranes for the chargers.

Erection was carried out above the old soaking pit ilding and through the existing mill roofs without erference with continuous steel production below.

An account is given of the factors influencing the ıal unusual design which makes use of heavy portal ames and Vierendeel girders, and the erection pro-dures are described.

The Control Room, spanning 90 ft. across the new akers and carrying live loads from the pit super-structure entailed further design problems, the solutions of which are outlined.

Mention is made of the foundation work for the main building and for the new soaking pits.

Introduction

Samuel Fox set up a wire drawing business in the South Yorkshire village of Stocksbridge in about 1840. Since that time almost continuous development has taken place with the result that to-day, the firm of Samuel Fox & Co., Ltd., a branch of The United Steel Companies, Ltd., is one of the principal producers of high-quality alloy steels in the United Kingdom, employing over 6,000 workpeople. The steelworks are situated in the narrow and steep sided valley of the River Little Don, about ten miles from Sheffield, and development over the years, with these limitations of space and under varying conditions of production, has resulted in extremely congested areas.

Fig. 1.—Old soaker gantry

The Billet Mill was established on its present site in 1905 and is steam-driven, while the reheating of ingots prior to rolling has been carried out since that time with very few modifications in the existing coal-fired soaking

*Paper to be read before the Institution of Structural Engineers 11. Upper Belgrave Street, London, S.W.1, on Thursday, Novem- 26th, 1953, at 6 p.m.

pits. Whilst the mill is considered to have many more years of life, the soaking pits had so deteriorated, the original cast-iron casings being cracked, burnt and generally useless, that the cost of repairs and maintenance, together with a most extravagant consumption of best quality coal, were factors which influenced the management's decision soon after the end of the war to install sets of new soaking pits. The two ingot chargers serving the old pits together with the gantry and roof structure had similarly deteriorated, so that a complete replacement was required. Reconstruction would require to be carried out at the existing site owing to limitations of space.

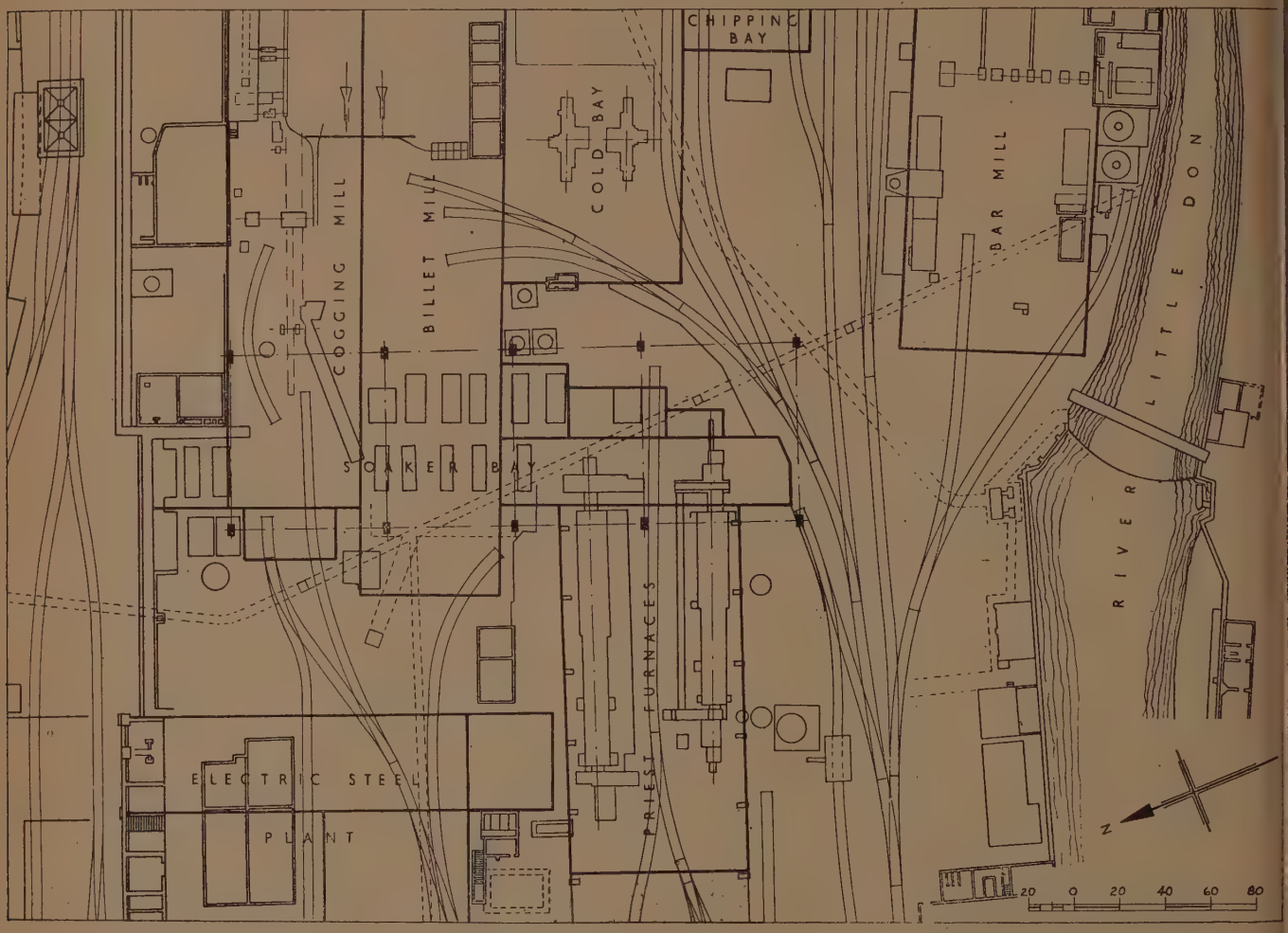

Fig. 2.—Location of site

Existing Conditions

Fig. 1 gives a general view of the old soakers from the north end of the existing shop, and the general poor condition of the pits and gantry structure is illustrated. Continuous trouble with the crane tracks, which had been known to move as much as six inches, had been experienced for many years and the presence of various stiffening members and backstays, interfering with the safe operation of the chargers, was one of the many difficulties experienced during reconstruction.

A general site plan showing the existing layout with relation to adjacent shops and railtracks is given in Fig. 2, and it will be seen that the crane gantry serving the old soakers was sited on a roughly north-south axis, running up to a retaining wall at the north end. This gantry was roofed over for the whole length, the trusses spanning across roof legs from the gantry at the south end, whilst at the opposite end, cover was mainly provided by two 60 ft. span roofs, these running at right-angles to the soaker gantry and being extensions of the Billet Mill roofs.

Two Priest continuous furnaces are sited within a steel-framed building on the west side, connecting at right-angles with the old soaker bay. Hot ingots are extracted from these furnaces before charging into soaking pits, and this building was therefore required to be maintained in the final scheme.

The area of the soaker bay was thus enclosed by adjacent buildings on both the east and west sides and at the north end by a retaining wall. At the south end the works yard was crossed by several rail tracks leading into the mill and these are occupied almost continuously by important traffic.

The existing soakers were hand-fired through the tops of the fireboxes and ashing out was done from a pit (known as the soaker cellar) some nine feet below general yard level on the west side. The flues from each pit were taken away on the opposite side, passing through a set of waste heat boilers before connecting to three 90 ft. high steel chimneys.

Numerous culverts and drains crossed the area and there was a considerable number of cables and service pipes, both above and below ground.

Design Requirements

The required layout is shown by Fig. 3 from which it will be seen that a battery of six new producer gas-fired

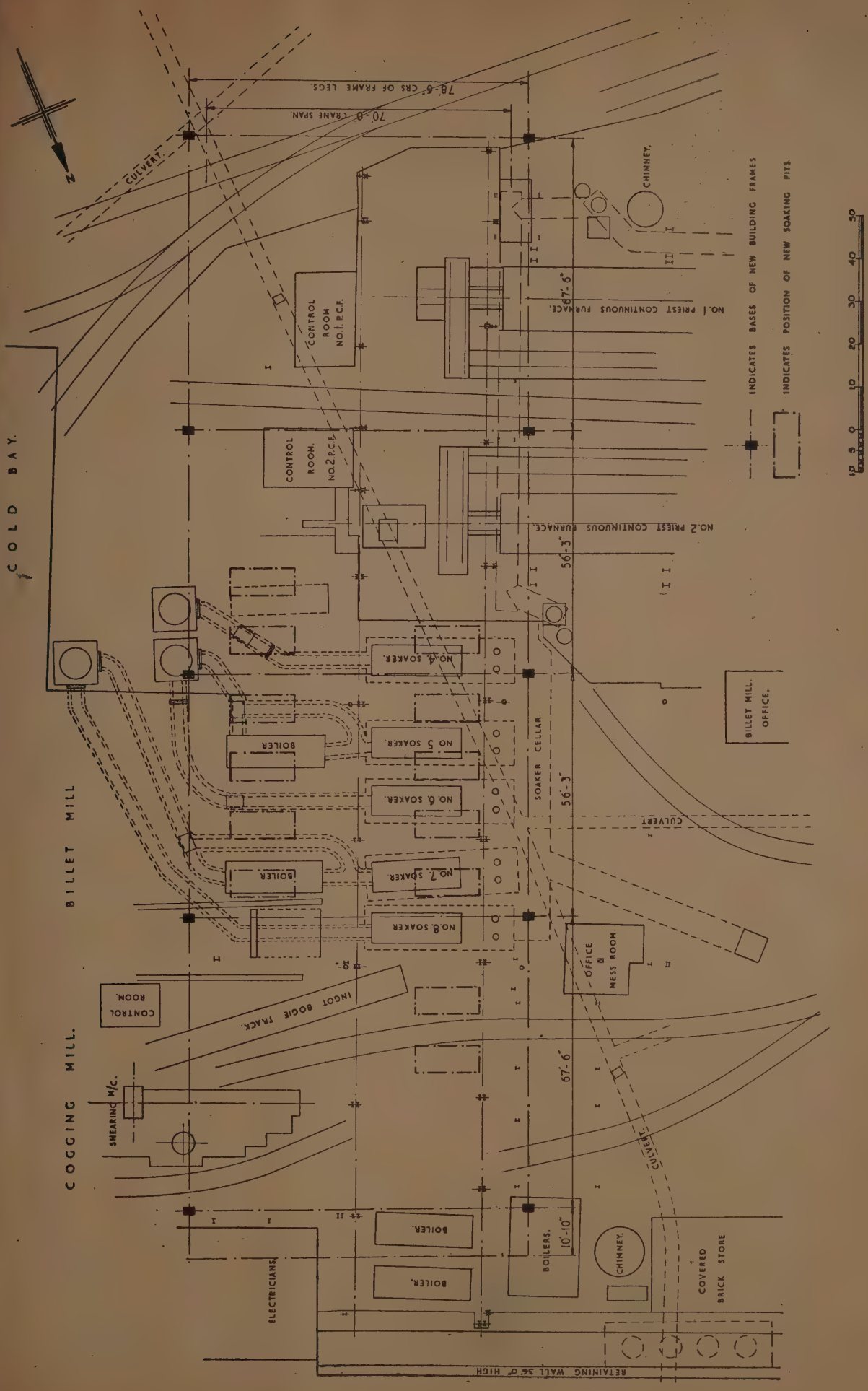

Fig. 3.—Site plan

pits were proposed, sited opposite the existing soakers in the area then occupied by the flues and waste heat boilers. The precise locations of these pits were actually arrived at after a detailed site survey and the subsequent determination of the stanchion centres for the new building is described below. A second battery of pits was to be installed on the opposite side after the completion and satisfactory operation of the first battery. This would permit the removal of the old soaking pits with no interruption to the reheating and rolling of ingots. Finally, central ingot bogey tracks and transfer tracks were to be provided running from the Priest continuous furnaces between the batteries of pits to the cogging mill end.

The level of the bottoms of the new pits was to be closely comparable with that of the old soakers, thus enabling the new ingot chargers when installed to serve both the old and new pits during the necessary trial and changeover period.

Thus, the general design problems with respect to the main building, which were presented to the Consulting Engineers in 1951, when the scheme was finally sanctioned, were briefly as follows :—

1. A 70 ft. span crane gantry to accommodate two 5-ton vertical ingot chargers each equipped with a 20-ton auxiliary hoist was to be provided suitably disposed above the new soaking pits.

2. Hand-operated cranes to service the chargers were to be provided at high level in parking bays at each end of the gantry.

3. A monitor type roof was to be provided, with large ventilators and adequate access for maintenance.

4. Provision was required for the support of brick-lined gas mains serving the new soaking pits from gas machines which were to be located adjacent to the retaining wall at the north-west corner of the building.

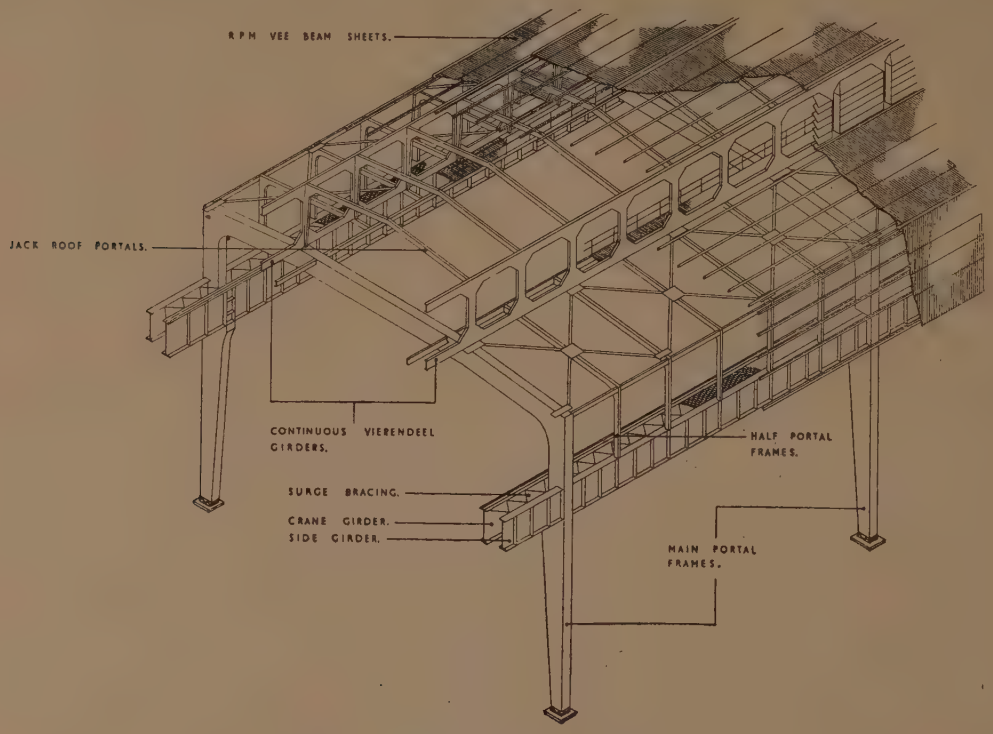

Fig. 4.—Sketch of new structure

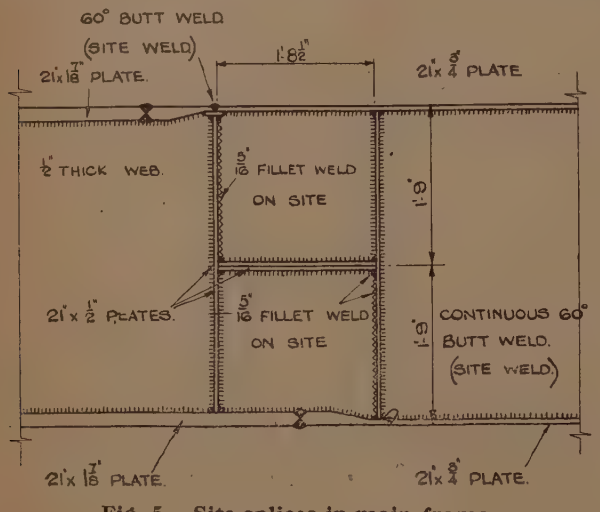

Fig. 5.—Site splices in main frames

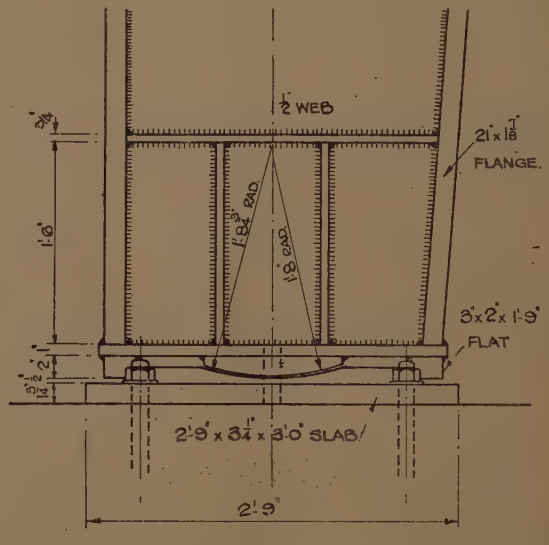

Fig. 6.—Base details

(At this stage, the mains were intended to run below crane girder level, but were subsequently re-sited at eaves level as explained below.)

5. Messrs. Fox wished to have a modern building of clean lines, preferably of welded construction. In view of the dust and fumes which would exist in this location,

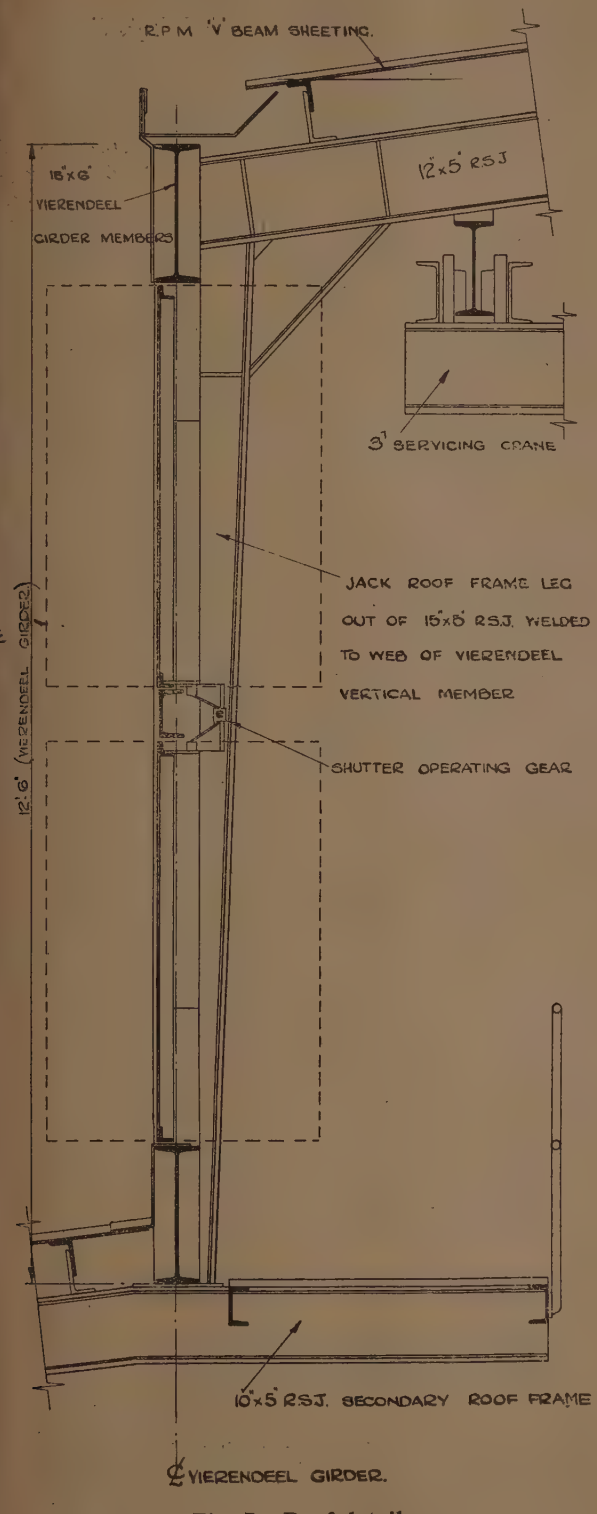
R.P.M. 'V' BEAM SHEETING.
15"x6" VIERENDEEL GIRDER MEMBERS
12"x5" RSJ
3t SERVICING CRANE
JACK ROOF FRAME LEG OUT OF 15"x6" RSJ. WELDED TO WEB OF VIERENDEEL VERTICAL MEMBER
SHUTTER OPERATING GEAR
12'6" (VIERENDEEL GIRDER)
10"x5" RSJ. SECONDARY ROOF FRAME
℄ VIERENDEEL GIRDER.

Fig. 7.—Roof detail

lattice girders and triangulated roof trusses were to be avoided as far as possible to prevent excessive lodgment of dirt and permit easy painting and maintenance.

6. The entire structure was to be erected through and above the old roofs with no interruption to the continuous operation of plant below. The mill would shut down only for the short Christmas and Easter holidays and for the annual works stop period during the summer.

7. Erection was to commence during the summer of 1952, and the dismantling and replacement of the existing cranes and gantry by one of the new chargers, followed by trials and the changeover to the first battery

Fig. 8.—North gable part completed

of new soaking pits, was to be programmed for the works stop period at the beginning of August, 1953.

Survey and Site Investigation

No accurate plans of the area existed nor were complete records available of the flues, culverts and services below ground, and apart from the Priest furnace building, which is a relatively recent structure, there were no drawings or details of the existing roofs.

A comprehensive three-dimensional survey was therefore carried out and although considerable difficulties were imposed by the general congestion, and lack of sight lines, together with the almost continuous traffic of hot ingots about the area, an accurate result was obtained. This accuracy was of great value, enabling the new structure and erection procedures to be planned on paper with no difficulties from dimensional errors arising during the subsequent construction.

Fig. 9.—Gas Mains

Previous excavation for foundations in this area had shown the presence of a bed of hard shale dipping from north to south towards the river. Trial holes were put down in the south-west corner of the area, adjacent to the site of an old dam and the original river bed, which were filled in when the river was diverted and the yard

level raised. Here, the shale bed was proved at some 16 feet below ground level, below filled material, silt and soft clay. It was decided that all the major loads could be founded without undue difficulty on the bed rock.

Development of Design

It was obvious at the outset that very few locations would be available for structural supports to the ground, since these would require to have adequate clearance from all railtracks and plant, and to be sited to permit the retention of the existing buildings and crane tracks until completion of the new shop.

Thus one of the chief features of the design was to be the spanning of the heavy crane girders and roof structure over relatively large distances. From this consideration, and having in mind the required profile, with the provision of clear openings for vertical ventilators in a large monitor roof, the general scheme developed into the structural form illustrated in Fig. 4. It will be seen from this sketch that with portal frames spaced at wide intervals and carrying the crane girders, the support of the roof structure is shared between continuous Vierendeel girders spanning across the tops of the frames, and longitudinal girders situated alongside the crane girders. Thus the high level jack roof is supported across the top booms of the longitudinal Vierendeel girders, while the lower roof and side sheeting is carried by half portals connected between the lower booms of the Vierendeels and the top flanges of the side girders. In this way the required profile was obtained, and the other conditions fulfilled, with a complete absence of unsightly secondary members across the roof space.

In order to obtain lateral stability for the high level roof, it was decided that this structure would be designed as a series of portal frames, the verticals of which would be combined with the intermediate posts of the Vierendeel girders, suitably stiffened in the direction across the roof span. The end bays at each end of the building were designed to support 3-ton hand-operated servicing cranes running on tracks suspended from the high level rafters. Diagonal bracing in the plane of the lower roof rafters was introduced to provide horizontal girders designed for side wind forces and to restrain the lower booms of the Vierendeel girders against the outward thrusts from the jack roof portals.

Siting of Main Frames

Having decided on the general structural arrangement, it was necessary to investigate and determine the locations for the main frames, so that the detailed design could be carried out.

A preliminary investigation had suggested that these would be spaced at intervals of the order of 100 feet or more, but it was considered that such large spans would greatly increase erection difficulties having regard to the probable size of girders and the available access to the site. Apart from the question of maintaining clearances to existing plant and rail tracks, further difficulties were imposed by the extremely limited space available for the accommodation of the two new batteries of soaking pits. This limitation meant that the required number of pits would need to be spaced at close intervals and it was originally thought that this might preclude the siting of any building foundations within the areas thus occupied. However, by disposing the soakers at the absolute minimum centres to permit operation of the covers, it was finally made possible to site one of the frames within the area of the new soakers.

The siting of the main portal frames and their foundations which was finally adopted is illustrated in Fig. 3, and it will be seen that a symmetrical arrangement was obtained, there being two end spans of 67 ft. 6 in., and two interior spans of 56 ft. 3 in., with a common denominator of 11 ft. 3 in., for the spacing of the intermediate roof frames and the verticals of the Vierendeel girders. This arrangement was the outcome of very considerable investigation by trial and error, proposals being set out on paper followed by a closer inspection of conditions at site, until the solution with the least number

Fig. 10.—Frame base in soaker cellar

f difficulties was found. Although all these locations resented problems, e.g., two were in the old soaker ellar, one was within the area of an existing chimney nd another directly over the intersection of two culverts, he arrangement was subsequently justified by the bsence of any intereference with steel production during onstruction. Consideration was given at this time to he question of possible fixity at the bases of the main ortal frames, and it was decided, in view of the small pace available in each position that it would not be ossible to provide satisfactory fixed ends. Rocker oints were therefore adopted and these were established t the level of the bottoms of the new soaking pits,

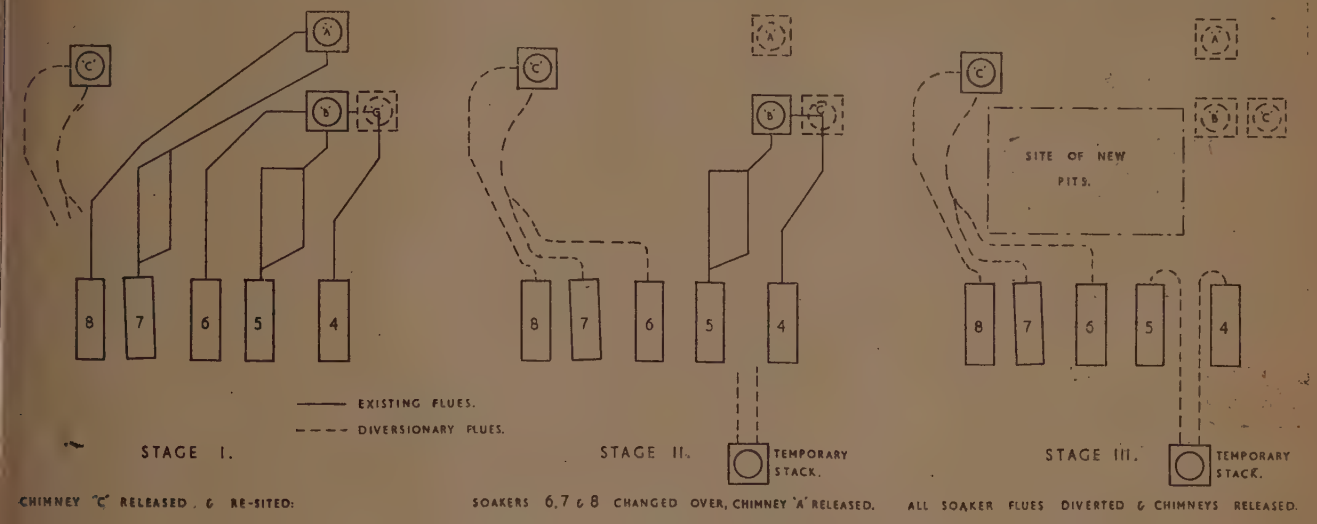

Fig. 11.—Diversion of flues

which level is some three or four feet below the existing shop floor and yard. The foundations generally consisted of mass concrete blocks taken down to the rock.

Main Frames

The main portal frames, which were analysed by strain energy methods, consist of 21 in. wide flange plates varying from 2 in. thick to $\frac{7}{8}$ in. thick with a $\frac{1}{2}$ in. thick web plate. Above crane girder level the depth of the section is constant at 3 ft. 6 in. with a radius of 4 ft. 6 in. for the curved knees, while the tapered legs vary from overall depths of 4 ft. 6in. at the crane brackets to 2 ft. 6 in. at the bases. The flanges are welded continuously to the web using $\frac{3}{8}$ in. fillets and $\frac{1}{2}$ inch flat stiffeners are provided. The site splices were detailed as shown in Fig. 5, and illustrated in Fig. 13 and were designed to permit accessible downhand site welding for the single vee butt welds in the flanges.

The detail of the bases of the frames is given in Fig. 6. It will be seen that the rocker type of seating is provided, the radii of the machined bearing surfaces on the baseplate and frame leg being 20.75 in. and 20 in. respectively. Keeper flats were welded to the end plate of the frame as shown. The bearing surfaces were greased before erection, and where the bases were situated below the final ground level, brick pits were constructed, filled to 3 in. above the bearings with a flexible bituminous compound. The frames were then sheathed by expansion jointing to ground level and the remaining space filled with concrete.

Crane Girders

Since all the main frames were founded on the same shale bed no appreciable differential settlements were anticipated and it was accordingly decided that the crane girders would be designed as continuous spans with site welded joints over the supports.

These girders are 6 ft. 6 in. deep with single continuous flange plates, 24 in. wide varying in thickness from $\frac{7}{8}$ in. to $1\frac{5}{8}$ in. The $\frac{1}{2}$ in. thick web is stiffened by intermediate 6 in. × 5 in. and 6 in. × $4\frac{1}{2}$ in. R.S.J.'s with $\frac{1}{2}$ in. flat stiffeners provided at the supports, where a stepped joint site splice was provided, similar to that for the main frames.

Consideration was given in adopting working stresses for the design of the crane girders to the question of fatigue strength. Useful information in this connection was obtained from the Report of Committee 15—Iron and Steel Structures of the American Railway Association (published in Bulletin 489, October, 1950).

Further welded plate girders are provided, spanning between the webs of the main frames, and situated alongside the crane girders. These are designed to carry their share of the roof loading from the secondary "half portal" frames, together with the heavy loading from the gas mains which were finally sited centrally above the eaves gutters as shown in Fig. 9. These girders are 5 ft. 6 in. deep with $\frac{3}{8}$ in. and $\frac{1}{2}$ in. thick webs and flanges varying in thickness from $\frac{5}{8}$ in. to $1\frac{1}{2}$ in. Stiffeners consist of 6 in. × 3 in. × $\frac{3}{8}$ in. and 5 in. × 3 in. × $\frac{3}{8}$ in. angles provided with end plates to enable longitudinal welds at the junction with the main flanges.

The top flanges of the crane girders and the roof girders are at the same level, and are connected by a braced girder and chequer plating, the whole system providing the necessary lateral resistance to crane surge and wind. Thus a generous access and maintenance walkway is provided for the cranes, this having 3 ft. 5 in. clearance between the main frames while the crane clearance at the main frame positions is 12 inches. A further chequer plated walkway is provided at the level of the lower crane girder flange and the space or "tunnel" between the two girders provides a convenient and accessible duct for cables and other services.

The crane rails are of 110 lb. per yard solid section and are fastened by clips, machined to the profile of the lower flange of the rails and riveted to the crane girders with no clearance. The clips were delivered to site undrilled, the holes being marked off and drilled after lining up the rails. Rail joints were scarfed at 45 degrees and Thermit welding was carried out at site before fastening down. The chief factors for consideration in this connection were the maintenance of accurate line

with smooth running joints, and easy replacement in a situation where there would be continuous operation of cranes with a consequent relatively high degree of wear.

Roof Members

The Vierendeel girders span continuously across the tops of the main frames and are relatively lightly loaded, although provision is made for the 3-ton servicing

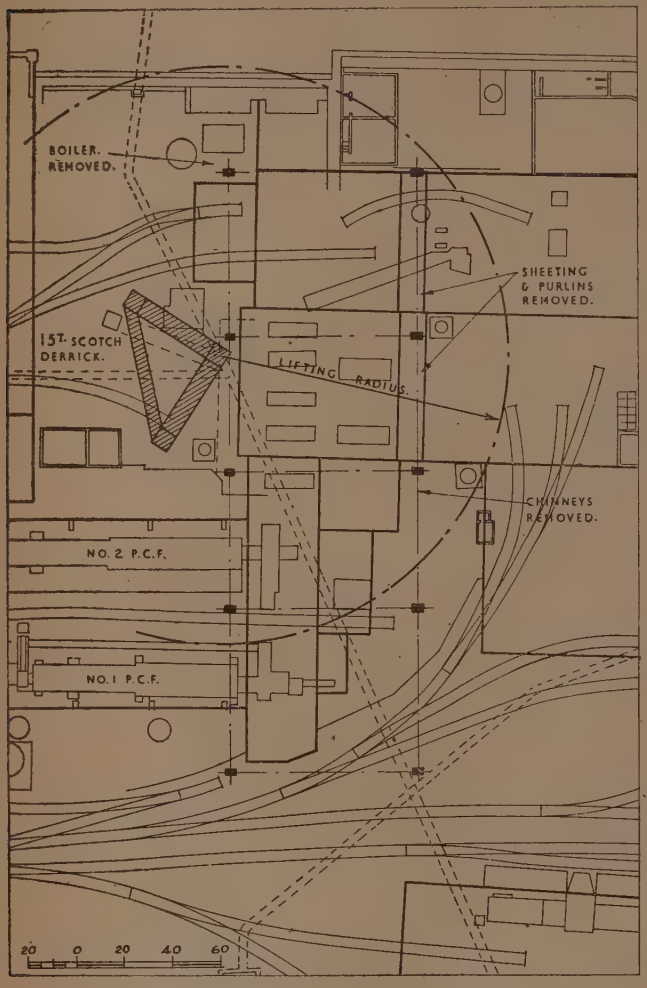

Fig. 12.—Siting of erection crane

cranes. The spacing of 11 ft. 3 in. which was adopted for the intermediate posts was maintained for the spacing of the verticals and an 18 in. × 6 in. R.S.J. section was finally adopted for the chords and verticals. It was considered that the small economy which might have been obtained by varying the sections for these girders would be offset by the loss of standard-sized openings for ventilators together with poorer appearance.

Stepped joints in the booms were adopted as site joints, positioned in the panels immediately adjacent to the supports, and the actual locations of these were finally settled after erection methods had been established.

The jack roof, spanning 37 ft. 6 in., is composed of 10 in. × 5 in. R.S.J. cranked beams and split 15 in. × 5 in. R.S.J.'s tapered from 10 in. to 5 in. were shop-welded to the webs of the 18 in. × 6 in. R.S.J. verticals of the Vierendeel girders to provide the vertical legs of the portal frames. The knee joints, fabricated out of rolled sections were also shop-welded to the main girders and a bolted site connection with the rafter beams was provided. This detail is shown in Fig. 7.

The lower roof members were composed of "L"-shaped 10 in. × 5 in. R.S.J.'s with a curved knee joint and connected to the underside of the bottom boom of the Vierendeel girders. The ends of the horizontal legs were cantilevered 4 ft. inside the building to provide a 3 ft. 6 in. wide access walkway at high level, while the vertical legs were bolted to the top flanges of the side plate girders. The roof slopes are almost flat, and in view of the traffic which was anticipated during the erection of the gas mains and subsequent maintenance it was considered that a rigid form of roof deck would be desirable, and 22 g. R.P.M. V-beam sheets were adopted.

The gas mains, which are brick-lined steel tubes varying in outside diameter from 8 ft. to 4 ft. 6 in., supply producer gas to both new batteries of soaking pits and to the two Priest Continuous furnaces. They are thus disposed along the whole length of the west side of the building, across the north gable and along the major part of the east side. The final layout of these mains and the decision to locate them above eaves level was only possible at a relatively late stage but the necessary modifications were effected without serious delays to the fabrication and erection programmes. The method of supporting the mains is shown in Figs. 8 and 9.

In order to provide sufficient parking space for the chargers at the north end of the shop it was necessary to cantilever the crane girders and roof girders for a distance of 11 ft. 3 in. beyond the last main frame at this end. No space was available for putting down supports for the gable framing here, owing to the presence of boilers and other plant in the area below

Fig. 13.—Erection of a main frame

and it was decided to design the north gable frame as a deep lattice girder spanning between the cantilevered ends of the crane and side girders. Owing to the presence of the large jack roof the gas mains had to be cantilevered out from this girder, supported on horizontal tie beams with raking struts taken back to the verticals of the gable girder. It was necessary to stiffen the bottom boom of this girder against the

horizontal components from the rakers and thus a braced walkway was provided at crane girder level across the gable, connecting the two longitudinal crane access walkways.

Further decking is provided across each gable at high level connecting the longitudinal jack roof walkways and there is interconnection between these, the crane walkways, and the external system giving access around and above the gas mains.

Construction of Foundations

Work was commenced on the foundations on Christmas Eve, 1951, when the two bases located in the ashing out cellar immediately in front of the soaking pits were constructed up to the cellar floor level.

Excavation commenced at noon on Christmas Eve, when the soakers were still hot and the work completed at 6 a.m. on December 27th, when firing was commenced. The level of the base plates to the main frames is approximately 1 ft. 6 in. above the level of the existing soaker cellar and in order to avoid interruption at this stage to the ashing out of the soakers, these two foundations were not constructed up to the final level. A dovetailed slot was provided in the concrete, with provision for the holding down bolts, and after filling up the recess with sand, the floor slab was replaced together with the tracks for ash skips. These bases were subsequently completed soon before erection commenced, after certain modifications had been carried out to the fire doors of the two pits concerned to permit their continuing operation after erection of the frames. Fig. 10 shows one of the completed bases with the frame leg in position. The existing soaker fireboxes can be seen in the background and also visible is one of the sheet steel covers which were provided to protect the foundation and base of the frame from the heat, fumes and dirt.

The other foundations were able to be constructed during normal production periods although in almost every case difficulties were presented. Underpinning of adjacent foundations and modifications to rail tracks and adjacent structures were usually necessary. One of the foundation blocks was constructed below an existing steel chimney, the base plate of which was underpinned.

In order to construct the foundations for the new soaking pits on the east side and for the frame legs in that area, it was necessary to divert the flues from the existing pits.

Originally, the five flues from the existing soakers were taken to three chimneys, two of which received two flues each while the remaining flue was taken to the third chimney. This arrangement can be seen in Fig. 3. Of these chimneys, two were to be required in the final layout sited together with a single new waste heat boiler about 50 feet away from their original position. A scheme was evolved which enabled the old flues to be diverted in stages so as to maintain full time operation of the soakers. The various stages are illustrated in Fig. 11 and briefly, the system was first to release the single flue chimney and to move this to a temporary position inside the mill. Diversionary flues, collecting waste gases from three of the five soakers were then constructed connecting to this stack and the changeover was made by working continuously at a stop period. Similarly a temporary stack was erected on the west side to receive the diversionary flues, which reversed their original direction, from the remaining pits. Thus, after the Easter stop week-end of 1952 all flues were diverted with the area of the new soakers clear for excavation and with two chimneys available for resiting. Before the stacks were moved to their new positions the brick linings were removed and the uppermost plate sections taken out to improve stability during movement, and to allow the one chimney to pass below an existing valley beam into its temporary position inside the mill. There was only one possible location for this particular stack and this was to give rise to some further difficulties during the construction of the Control Room, which had to be overcome at a later date.

Fig. 14.—View from the north on August 6th, 1952

All the foundations for the main building were completed according to programme in the early summer of 1952. The setting out of the frame centres and holding down bolts was a somewhat difficult operation requiring extensive use of the theodolite and numerous geometrical expedients to overcome the almost complete absence of sight lines and general obstructions. These operations were successful and there were no subsequent troubles from misalignment.

Erection of Main Building

A schedule of weights and dimensions of all the principal items of steelwork was prepared at an early date and site meetings held with the erection contractors. One of the first conclusions reached was that it would be desirable to lift the main stanchions and girders down through the existing roofs from above, especially in the important area at the north-east corner, adjacent to the Cogging Mill and crop end shears. The handling of 70-ft. long girders into the mill at this area and lifting up from the shop floor, whilst not impossible, would certainly increase the risk of interference with production. A further important factor was the level of the crane girders which fortunately was above that of all the existing valley beams so that when lowering into position from above. it would only be necessary to remove sufficient sheets and purlins as to provide a narrow slot in the line of the girders along the roofs. Thus, it was decided to provide a 120 ft. jib 15-ton scotch derrick erected on gabbards 40 ft. high.

Consideration was next given to the siting of the derrick, and there appeared to be two alternatives, one being somewhere in the relatively open area between the old building and the Electric Steel Plant, and the other in the similar area adjacent to the soaker chimneys at the south-east corner. The lifting range of the derrick was investigated for both locations, making use of a cardboard template on the $\frac{1}{8}$ in. scale survey plan and the finally selected siting at the north-west end, is as shown in Fig. 12. The factors which influenced the decision are as follows :—

1. The greater and more difficult part of the structure from the erection viewpoint, would be covered by the crane.

2. It was considered that the steelwork at the south end which was out of range could be handled by derrick poles without undue difficulties.

3. The larger number of accessible rail tracks in this area suggested that some use could be made of Messrs. Fox's own steam cranes.

4. The alternative site would in the near future be required for the construction of boiler foundations and flues.

Access to the site for the steelwork was also a matter for consideration and the precise alignment of the scotch derrick was influenced by the necessity for the jib to cover the rail track running from the Electric Steel Plant into the mill. The nearest areas available for stocking the steelwork were selected at various points up-works and it was necessary to carry the heavy items down to the site by steam crane, as and when required for erection. The somewhat tortuous routes to the site were investigated to check clearances and head room where the rail tracks passed through shops or below pipe gantries, etc.

Fig. 15.—View from north-east on August 15th, 1952

Special consideration in this connection was given to the large Vierendeel girders. These were 12 ft. 6 in. deep overall and while the booms were only 6 in. wide, the shop-welded legs of the jack roof portals with the projecting stubs of the corresponding rafter members increased the overall width to 4 ft. 6 in. It was considered desirable that these girders should be delivered to site and erected immediately in such lengths as would span between the main frames, since there was insufficient space adjacent to the site of the crane for the setting up and welding of smaller units prior to lifting. Nor did this operation appear possible without undue interruption to rail traffic in the area. Thus the longest length of girder, which included the cantilever portion, was about 80 ft. British Railways were unable to deal with a girder of these proportions but it was found that road transport from Scunthorpe would be possible, and this was arranged. The girders were offloaded at the works by two steam cranes and carried to the site in the vertical position during selected periods of minimum traffic. Since they were extremely slender laterally, temporary stiffening was introduced, and the greatest care had to be exercised during carriage to the site and

ting. Although this method was not without difficulties and there were in fact several anxious moments, ie desired result of minimum interference with traffic id production was obtained.

The main feature of the erection programme was that ie more difficult and inaccessible lifts were to be ranged for Messrs. Fox's "stop week" in August, 952, although it was intended that erection would, if ossible, commence before this time. Accordingly, the otch derrick and the gabbards were delivered to site June and erection of these items commenced during at month. The finally selected position for the crane ntailed the siting of the king post for the derrick mmediately in front of the existing soaking pits over he ashing out cellar. The roof of this cellar was omposed of steel joists and floor plates spanning between he outer wall and the front castings of the soakers. As as been mentioned, the pits were in very poor shape, nd this existing steelwork, in addition to supporting he shop floor over the cellar, was buttressing the front astings which were badly out of plumb. It was herefore necessary to provide independent support for he king post gabbard and this was arranged to avoid ny possibility of load on the casings to the existing oakers. The remaining two gabbards were constructed vithout difficulty although some modifications were ecessary to adjacent roof structures. Kentledge in he form of 50 cwt. ingots was provided, and from time o time during erection, as special casts were required, certain ingots were removed for rolling and replaced. Erection of the gabbards and crane was completed in three weeks and by July 16th, 1952, the three components of the first main frame at the north end had been erected and site bolted. Fig. 13 shows the horizontal member of the portal lifted into position with site bolting in progress at the stepped joints. Also visible is the slot provided through the Billet Mill roof to permit the lowering of the crane girders. The main frames, crane girders and roof girders were then erected in sequence from the north end, and there was good progress during the first week in August, this being the works stop period when there was relative freedom for

Fig. 16.—Erection of frame leg by steam cranes

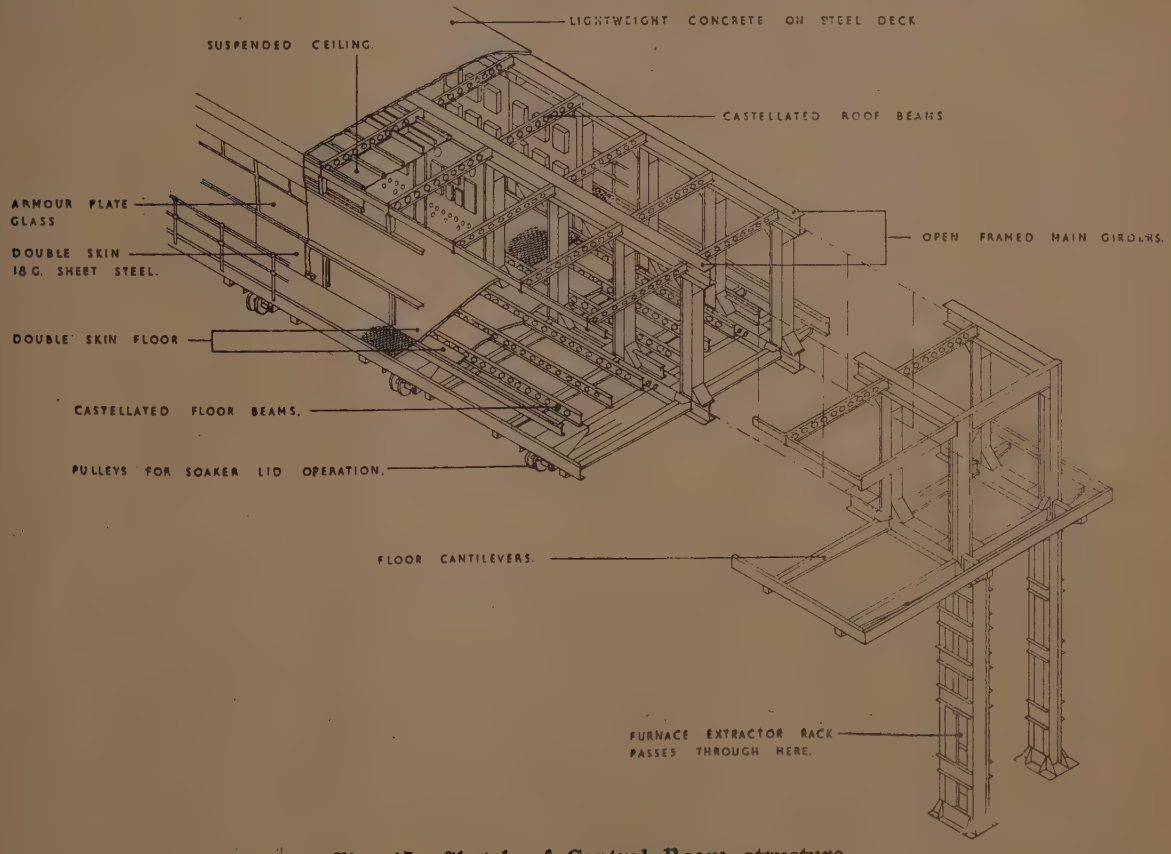

Fig. 17.—Sketch of Control Room structure

the carriage of steelwork to the site. The position at August 6th may be seen from Fig. 14 where three frames have been erected with the crane girders. The re-sited steel chimney receiving the diverted soaker flues can be seen in the left foreground, while the remaining stack of the original three is shown in process of dismantling prior to re-siting with the new waste heat boiler. The longest Vierendeel girders were delivered by road during this period and erected successfully.

Fig. 15 shows the situation at August 15th, approximately a month after the commencement of erection. Here, four of the five main frames have been erected with the Vierendeel girders and this photograph illustrates the limiting range of the scotch derrick for direct lifts. The secondary roof members in this area were dealt with by the crane, but care had to be exercised to ensure the correct sequence to enable the jib to be lowered sufficiently to obtain the future long-range lifts. Thus the span of Vierendeel girder on the west side immediately opposite the scotch derrick was left out until the completion of all the lifts on the east side. This meant that the erection of the roof, which originally had been intended to proceed from north to south in sequence, was slightly interrupted. All the heavy longitudinal members, including the Vierendeel girders, had been arranged with site splices to suit this sequence, but it would in fact have been better to have arranged the joints in the west side Vierendeels to permit the insertion, as one of the last operations by the crane, of the section immediately opposite the king post. However, the erection of items by the crane proceeded sufficiently quickly to avoid any delays to the overall programme. The end portal frame at the south gable was erected entirely by steam cranes working from the adjacent sidings, but for the last spans of crane and roof girder at this end the scotch derrick was used to take the members over as far as the range would permit, and the final lifts were carried out jointly by the main derrick and a steam crane lifting at each end of the girders. The most difficult operation was perhaps that required for the erection of the crane and roof girders for the end span at the south-west corner, i.e., over the Priest continuous furnaces, and out of range for the scotch derrick. These two furnaces were roofed over and this roof would finally join up to the new shop. At ground level, the space between the two furnaces was occupied by a rail track used continuously for the shunting of ingot cars, and by the ingot bogey transfer tracks for the furnaces.

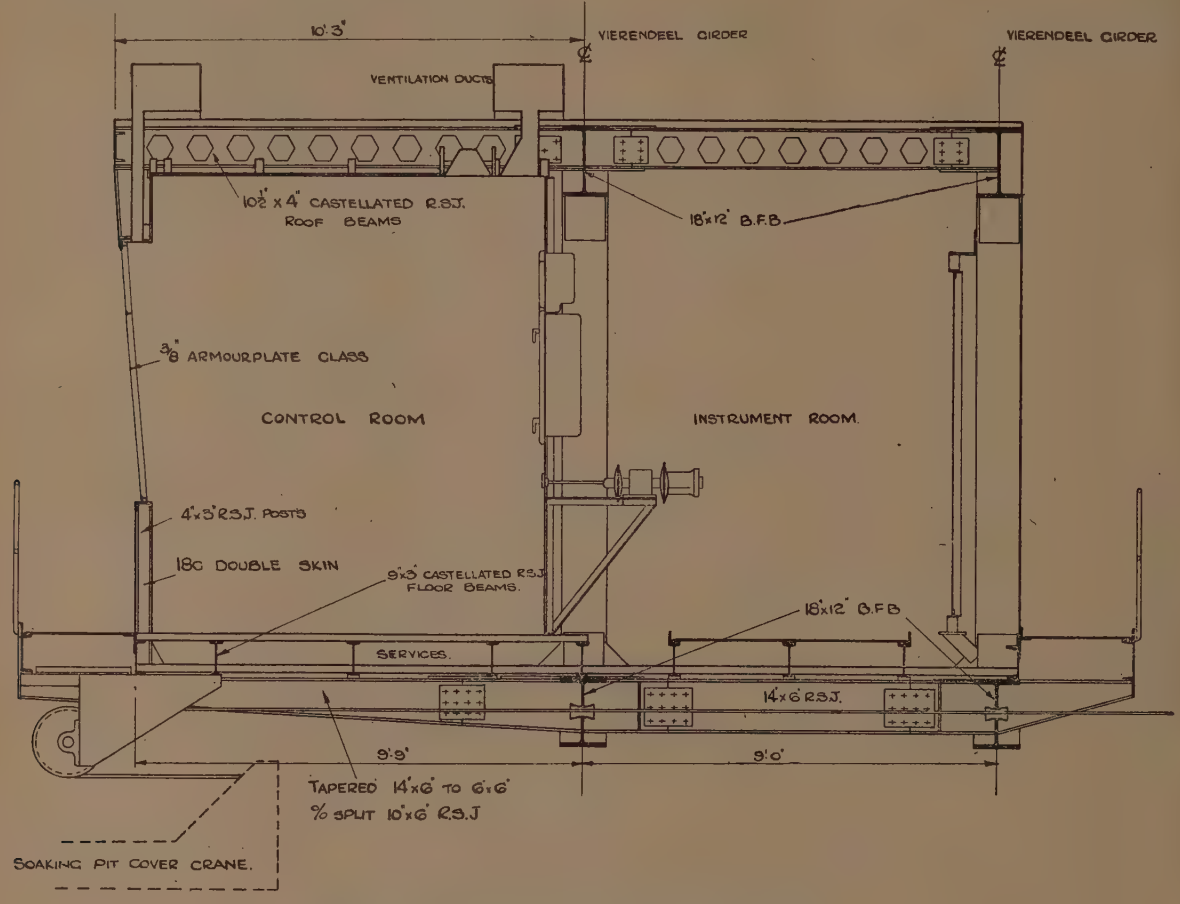

Fig. 18.—Cross section through Control Room

There was also a 5 ft. diameter gas main running below and parallel to the line of the new crane girders. These girders could not therefore be handled up from below and at first sight it appeared that a derrick pole would be required situated between the two continuous furnaces which would transfer the lifts from the main crane. This idea was, however, rejected on account of possible interference with ingot traffic below and it was finally decided to "float" the girders across from the north over the adjacent crane girder span, making use of the scotch derrick in conjunction with tackle fixed to the Priest furnace building, until the position was reached when the ends were accessible to steam cranes in the works yard. Fig. 16 shows the situation immediately after the erection of the frame leg at the south-west corner. The roof girder can be seen lashed back to the Priest furnace roof and held at the other end by the scotch derrick. One of the steam cranes was subsequently released from the frame leg and used to pick up the end of the roof girder and erect this in position in conjunction with the main derrick crane. A derrick

e was used to erect the Vierendeel girders at this l, a suitable location being possible.

By the end of August, the skeleton of the building d been erected, and the " filling in " of secondary roof mbers with purlins and sheeting rails was continued. e scotch derrick was dismantled at the end of Septem-r, and removed from site.

Soaking Pit Foundations

Excavation to a general level for the first battery of new soaking pits was commenced soon after the old es had been diverted and the site cleared, but the al plant details were not settled until September, 52, when the foundation drawings were issued to the ntractor. These foundations were finally completed the beginning of May, 1953, although the installation the pit superstructure was able to be commenced fore that date for three of the pits, together with the ebrick linings to the flues. The loads from the new aking pits, which are brick-lined and encased by steel ating, are taken by a row of R.S.J.'s below the bottom ating to a reinforced concrete slab, 1 ft. 10 in. thick, cluding a top layer, 6 in. thick, of refractory concrete. l the important foundations adjacent to the soakers, pecially those for the main building, were similarly otected by a 6 in. layer of refractory concrete. The ix adopted was one of crushed firebrick, grog and minous cement in the proportions of 3 : 2 : 1, and st cube results showed an average crushing strength 5,000 lb. per sq. in. at two days. Separate mixing ant was used for this work and care was exercised to oid contamination with Portland cement.

Certain underpinning work was again necessary, rticularly in the area of the adjacent Cold Bay where e gable stanchion carrying the end of the crane track this shop was temporarily supported and the existing undation removed to permit the construction of the s main downcomer bases. Subsequently an additional ft. length was spliced to this stanchion, which was ally founded at the lower level. A further difficulty this area was the existence of an important and avily loaded siding into the mill which was located mediately above the site of the main flue from the w soakers. This rail track was supported by R.S.J's d concrete piers suitably placed to enable the flue to constructed without interference to traffic. The undation work and construction of flues was completed time to permit the installation of plant to programme, d the first pit was fired to dry out the flues in July, 53. The foundation contractor experienced con-derable difficulties with regard to access to and from e site for concreting materials and removal of spoil, t the maximum assistance in this connection was ven by Messrs. Fox.

An electric scrapyard crane which had been replaced, s reconditioned and modified by Messrs. Fox to suit e span of the new gantry and was operated with a iling cable. This proved invaluable, both for the undation work and particularly during the installation the pits.

Control Room

The details for instrumentation and controls for the w soakers were finally settled towards the end of 1952 d the requirements in respect of the Control Room re briefly as follows :—

1. The structure was to be sited above the six new pits the east side of the new shop, as far into the shop as ane clearances would permit.

2. Thirteen instrument panels (one for each pit in the al installation) each 5 ft. 4 in. wide, were to be accom-odated in line, with space at the Cogging Mill end for a ogramme office. These panels form a central longi-tudinal partition with a 9 ft. wide room at the back giving access to the instruments, and a similar room at the front of the panels from which the various intstruments could be read and the controls operated.

3. The maximum visibility both into the soaking pits and to both ends of the new shop was to be obtained from the control compartment, and an external access walkway was required at the front to enable further visual inspection of the interiors of the pits. Access was also required around the back of the structure.

4. A series of six pulleys were to be mounted at the front of the Control Room floor, one on the centre line of each pit. These pulleys were required in connection

Fig. 19.—Interior of Control Room during erection

with the lid operating mechanisms and there would be a resultant horizontal thrust towards the rear of the structure of 5 tons acting at the centre line of each pulley.

5. The instruments required protection from crane vibration and this indicated that the Control Room would require to be isolated from the main building.

6. Attention was to be given to the insulation of the control compartment against heat and noise, together with suitable ventilation or air conditioning.

It was first necessary to settle the plan layout having regard to the overall length required for instrument panels, and to consider the availability of space for supports to the ground and for foundations. There were very great limitations in this respect since the whole area covered by the soaking pits was occupied either by flues below ground or by gas mains, valve gear, and operating plant above ground level. It was thus concluded that the Control Room would require to span completely across the new pits, a distance of 92 feet. Since the central longitudinal partition was to accommodate the rectangular instrument panels it seemed logical and convenient to design this as an open-framed girder, i.e., so that the panels could be fitted in the spaces between the verticals. The possibility of providing a Vierendeel girder at the front of the structure was considered but rejected on account of the probable size of the verticals and the consequent obstruction to vision. Apart from the importance of keeping the front wall clear of main structural members to give a continuous window, there was also the difficulty arising from the presence of the two main frame legs at 56 ft. 3 in. centres. These were to be isolated from the Control Room, and since it was intended that the front wall would project into the new shop as far as the inner faces of the main frames, any main girder at the front would have to pass either in front of or behind the frame legs.

These were some of the factors which led to the structural solution illustrated in Fig. 17 and it will be

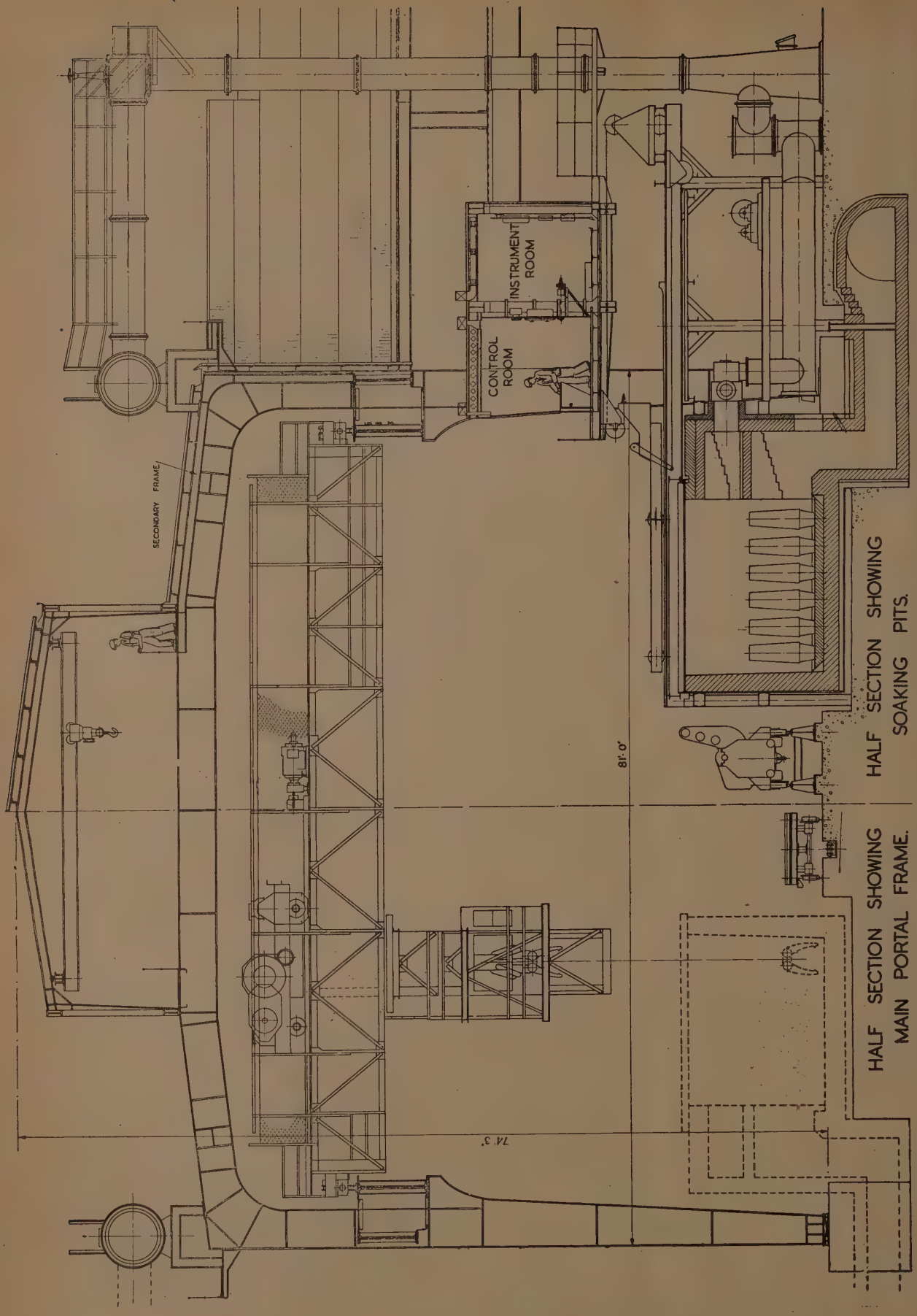

Fig. 20.—Cross section

een that two Vierendeel girders each of 92 ft. span are rovided, supported on rigid frames at each end. The entre lines of these girders were situated along the nstrument panel and the back wall respectively while he floor and roof members were arranged to span from he lower and top booms of the back girder, continuously cross the booms of the front girder and to cantilever ut 10 ft. to the line of the glazed front wall. In this vay, relative freedom was obtained to arrange the ontinuous observation window.

Use was made of castellated beams for the floor beams nd roof cantilevers, allowing for the passage and oncealment of services within the double skin floor and oof. The floor consists of two layers, each 2½ in. thick f lightweight concrete laid in conjunction with "Lewis" lovetailed steel sheets spanning between the lower and pper flanges respectively of the 9 in. × 3 in. × 12 lb. onstellated R.S.J. floor beams. The roof is of similar onstruction, having a single upper layer of light-weight concrete with a suspended ceiling of ⅝ in. thick coustic tiles. The walls to the Control Room consist f a double skin of 18 g. mild steel sheets, while for the nstrument room at the rear, a single skin of sheets was sed to clad the outside face of the second main girder. The floor cantilevers, and the lower chords of both main girders were braced together to provide a horizontal system transmitting the series of 5-ton thrusts from the soaker lid counterweight pulleys to the end frames. Rigid connections were provided at the junctions of the floor and roof members with the booms and verticals of the main girder to resist the torsion due to the external moment arising from the loaded cantilevered portion.

Certain of the structural details are indicated in Fig. 18, from which it may be noted that 18 in. × 12 in. broad flange beams were used for the booms of the Vierendeel trusses. The verticals were composed of two 12 in. × 4 in. channels with the toes in to form a box section and having 12 in. × 1 in. plates welded across the flanges. The flange plate thicknesses were increased for the end and penultimate verticals. Fig. 19 gives a general view of the interior of this structure before the instrument panels were fixed.

The drawings of the Control Room were issued to the steelwork contractor in January, 1953, and delivery commenced during the first week in May. It was essential that the Control Room should be erected sufficiently early to permit the installation of the instruments, together with the soaker lid operating gear, for which the pulleys were to be mounted below the Control Room floor. Owing to the headroom required, it was necessary to lift a section of the Billet Mill roof before erection commenced and this was accomplished without difficulty. The main girders were each delivered in two halves, a stepped site joint being provided in the booms. Access was again difficult but erection was carried out satisfactorily using two derrick poles, and making use of the soaker lid crane tracks which had by that time been erected and which provided a useful platform for setting up the girders, jacking to level, and site welding.

The steel frame of the Control Room was completed together with floor and roof to enable the installation of equipment and wall cladding to commence at the beginning of June.

A cross section through the Control Room is shown in Fig. 18 which also indicates the arrangement of the counterweight pulleys with the wire ropes passing through bell-mouthed tubes welded through the webs of the 18 in. × 12 in. broad flanged beams. It will also be noted that the observation windows, glazed with armour plate glass were canted forward to the same slope as the inner flanges of the main frames. Fig. 20 shows a general cross-section through the new shop indicating the relation between the soakers and the Control Room.

Conclusions

To the authors, this job has confirmed the advantages of welded rigid-framed steelwork as a structural medium for works of reconstruction in difficult locations. The parallel boomed open-framed girder, whilst not necessarily a more economical form of construction than the lattice girder as regards weight of steel, can provide a very convenient solution to structural problems where clear openings are required.

Also illustrated by this job, which was not without difficulties, and for which there was a relatively tight programme, are the great advantages obtained by the closest co-operation between all the parties concerned. An office was available near to Messrs. Fox's works, where the design and detailing was carried out, and from which the site supervision was based. This helped considerably in all aspects, particularly as regards progress, since it was necessary constantly to refer to existing conditions, and since there were of necessity numerous alterations of plant details from time to time.

Acknowledgements

The authors wish to extend their thanks to Messrs. Samuel Fox & Co., Ltd., for their kind permission to publish this paper. Acknowledgements are also due to Mr. O. Inman, M.I.E.E., Director of Development, Mr. C. M. Slocombe, A.M.I.C.E., M.I.Struct.E., Chief Engineer, and his staff, for their co-operation at all stages of the job.

The principal contractors were as follows :—

Steelwork—United Steel Structural Co., Ltd., and their sub-contractors, Messrs. Harry Peers & Co., Ltd. (fabrication of Control Room) ; Messrs. Cozens & Sutcliffe (erection) ; Messrs. Robertson Thain, Ltd. (sheeting). Foundations—Messrs. W. G. Robson, Ltd. Soaking Pits—Salem Engineering Co., Ltd. Instrumentation—Tinsley Instruments (London), Ltd. Painting—Industrial & Decorative Arts, Ltd. Ingot Chargers—The Wellman Smith Owen Engineering Corporation, Ltd. Three-Ton Servicing Cranes—Messrs. Herbert Morris, Ltd. Control Room Floor and Roof Decking—Steel Ceilings, Ltd. Control Room Cladding and Glazing—Messrs. Mellowes & Co., Ltd.

Book Review

Data Book for Civil Engineers, Vol. I Design, and Vol. II Specifications and Costs, by Elwyn E. Seelye, 2nd Edition. (New York : John Wiley & Sons, 1951 ; London : Chapman & Hall.) 11½ in. × 9½ in. Vol. I 521 pp., diagrams, tables. 80s. ; Vol. II 506 pp., diagrams, tables, glossary, 104s.

This is a completely revised second edition of a book intended to make easily available essential data in each main field of civil engineering. Important additions to Volume I include extensions to the sections on rigid frames and on soils and soil mechanics, a new section on aluminium, up-to-date airfield design data, standards for swimming-pools, solution of rigid frame bridge bents by influence line data, and design data for a large number of existing rigid frame bridges.

Volume II deals with the preparation of specifications for airports, roads, railways, bridges, dams, docks, drainage and sewers. It includes relative cost analyses for each type of work, and a classified glossary of terms.

A Derivation of Maximum Stanchion Moments in Multi-Storey Frames by Means of Nomograms*

By R. H. Wood, Ph.D., B.Sc., A.M.I.C.E., A.M.I.Mech.E., A.M.I.Struct.E.

Summary

A type of Nomogram originally devised by Dr. M. R. Horne for the solution of beam moments in rigid frames has been extended to the determination of stanchion moments and developed to include the effects of instability, and it has been found that this can be achieved without departing from the construction of intersecting straight lines. The solution for the behaviour of a stanchion in the elastic range and in the absence of sway can be seen by merely locating two points on the Nomogram and drawing a straight line, which straight line is the diagram of the restraint moments. The nomograms can be used to demonstrate with great rapidity the phenomenon of "unwrapping" leading to buckling. It is shown how to construct nomograms for continuous stanchions, and it is hoped that these nomograms will also be of use to those engaged in the more general elasto-plastic research into stanchion behaviour.

Introduction

An extensive investigation has been carried out by the Building Research Station into the elastic behaviour of rigid steel frames for multi-storey buildings, both in terms of bare frames and also of the stiffening effects of the cladding. A comprehensive rapid and economical design method for multi-storey frames has already been published[1] which made use of charts giving the influence of the worst possible arrangements of live load and dead load for both beams and stanchions, without involving the necessity of solving any equations whatever. Some degree of moment redistribution could easily be allowed for in the beams, whilst the treatment of the stanchions was based on a modification of the recommendations of the Steel Structures Research Committee[2]. An alternative method of evaluating the maximum beam moments by means of Nomograms was discovered by Dr. M. R. Horne and described by him in a paper published in THE STRUCTURAL ENGINEER in 1950[3]. These nomograms have the characteristic feature that a number of independent variables can be conveniently handled in one diagram, and they consist of straight lines throughout. The author has now developed this type of nomogram to evaluate the end moments in stanchions, and in so doing account has been taken of a feature of stanchion behaviour which is of importance, i.e., the change that takes place from acting end moments to restraining end moments as the direct load increases. This "unwrapping" of the stanchions has been noted in the investigations of the behaviour of stanchions in the plastic range[4], but it can also occur in the elastic range. This progressive change of sign of the end moments is associated with the concept of the "equivalent length" of the stanchion, and is automatically taken care of in the nomograms which are illustrated below. It is a matter of some curiosity that this type of nomogram can be made to cover the non-linear buckling effects of stanchions (in the elastic range) whilst retaining straight lines.

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The Use of Substitute Frames and Degrees of Restraint

Degree of restraint methods[5,6] and the use of elastic equivalents[7] have been used extensively in the solution of rigid frames. But when extremely rapid solutions are required for the worst possible arrangements of live loads, the use of close substitute frames is by far the best method of approach. Thus the "Draft Rules" of the Steel Structures Research Committee were based on a set of idealised single bay frames which served as a substitute for the frame to be designed, as far as stanchion moments were concerned. The author[1] used very extensive substitute frames, and the results could be read immediately from design charts to a considerable degree of accuracy, the ratio of stiffness of a member to total stiffness at any joint being the governing parameter. The nomograms devised by Horne were based on the substitute frame shown in Fig. 1, and this frame was found to be highly successful. Horne made use of the variables

$$Q = \frac{\text{Sum of beam stiffnesses}}{\text{Sum of column stiffnesses}} \text{ at a joint}$$

and r = ratio of fixed end moments on adjacent beams at any joint.

Provided the approximate relationship $\frac{K_2}{K_1} \doteqdot \frac{w_2 L_2/12}{w_1 L_1/12} = r$ was assumed between the stiffnesses and the fixed-end-moments then the beam moments were obtained from straight line nomograms where the straight lines denote the parameters Q and r at each end of the beam in turn.

It is proposed to use the substitute frame of Fig. 2 for the determination of stanchion moments. Here C_B is the clockwise out-of-balance fixed end moment (F.E.M.) due to combined live and dead beam loads at the end B, and C_A is at the lower end A. Hence for symmetrical single curvature (Fig. 3) $\frac{C_A}{C_B} = -1$; for symmetrical double curvature $\frac{C_A}{C_B} = +1$. When the conditions are unsymmetrical the largest arithmetical value of C may be taken as $+1$, whence it becomes necessary to trace the behaviour of the column under possible values of $\frac{C_A}{C_B}$ ranging from $+1$ to -1.

Some designers would no doubt wish to take the "modified" stiffnesses of the adjoining beams according to the degree of fixity at the remote ends; these modified stiffnesses range from $\frac{3}{4}K$ to $1.0K$ depending on

whether the remote end is hinged or fixed. Similarly the fixed end moments C_B and C_A may be modified if desired by " carry-overs " from the remote ends. Also allowance can be made for semi-rigidity of the column-beam joint by altering the effective stiffness of the beams. But before any of these refinements are attempted it is as well to consider the following points :—

(i) Although the proposed method is a safe, simple, economical design, it is not a " collapse " design. A collapse design will eventually show still greater strength, and hence refinements are at this stage rather unnecessary. When instability is present, however, a

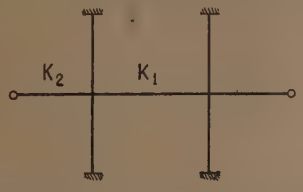


Fig. 1.—Substitute frame used by Horne

collapse design will be of necessity an elasto-plastic design, and none of the parameters involved in this analysis can be dropped, except in very special cases. Additional parameters will in fact have to be taken into account.

(ii) The Building Research Station has commenced an extensive study of the stiffening effects of floors, walls, encasement, etc. The first of a series[8] of " Studies in Composite Construction " has revealed the remarkable changes that can take place in the actual loads transmitted to beams that are supporting brick walls. Tests in the laboratory and on actual buildings have shown analogous effects in beam and floor combinations. This is shown quite clearly from a fundamental analysis of square slabs supported on flexible beams[9]. Thus in

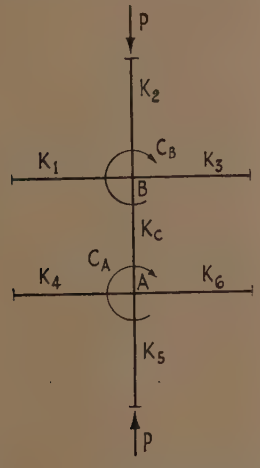


Fig. 2.—Substitute frame for stanchion moments

Fig. 4 a flexible beam is seen to shed a very considerable amount of load at the centre of the span (where it counts the most) and the effect on the bending moment diagram is pronounced. But that is not all, for the composite action of beam and slab may raise the effective stiffness of the beam against joint rotation by as much as 200 per cent.—which is of considerable importance in considering the buckling of stanchions. Coupled with this is the load dispersal *across* the floors to other beams in the vicinity, which has been detected in tests on an actual building frame at the New Government Offices, Whitehall Gardens. In due course it is hoped to publish the results but for the moment all that is necessary is to note that such refinements as " remote end conditions of beams " may not only be superfluous but decidedly misleading.

(iii) It is outside the scope of the note to discuss the probability of combined torsional-flexural buckling which is undoubtedly a possibility. But with the yield point as the criterion of design it is doubtful if this is a matter of practical importance except under rare conditions. The neglect of this phenomenon at this stage is certainly offset by the conservatism of items (i) and (ii), and also torsional buckling is impossible with encased stanchions. Torsional effects and the

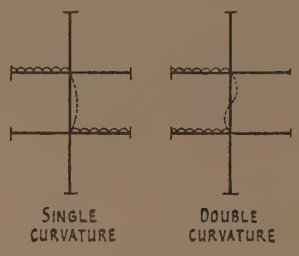


Fig. 3

warping of cross-sections may however become of some importance in a " collapse " design of bare stanchions.

Construction of a Nomogram when the Direct Load is very small $\left(\sqrt{\frac{P}{EI}} \cdot L = 0\right)$

Since there is no transverse load on the stanchion the nomogram is very simple. It would be sufficient to adopt a unit of clockwise, out-of-balance, fixed end moment C, applied at the top of the stanchion B (Fig. 2) and to vary the clockwise F.E.M. at A between $+C$ and $-C$. For the sake of symmetry, however, we will put rC at one end and $r'C$ at the other end of the stan-

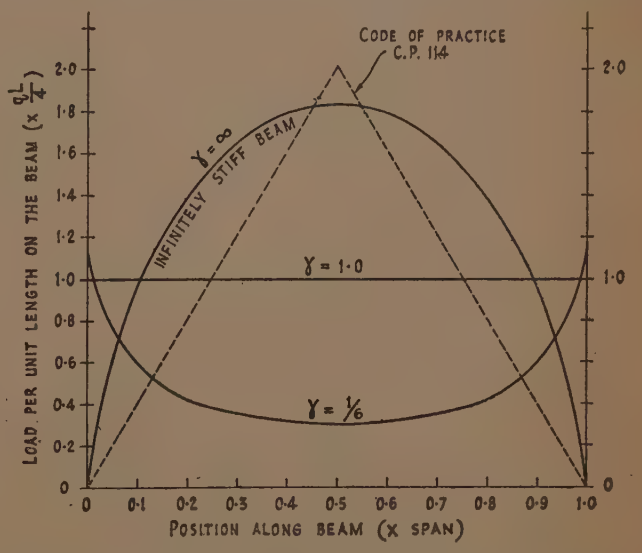


Fig. 4.—Distribution of vertical intensity of load on the beams supporting a square slab of side L. Slab carrying uniform floor load q.

$$\gamma = \frac{\text{Beam Stiffness}}{\text{Slab Stiffness}} = \frac{EI}{\frac{L}{D - \frac{}{2}}}$$

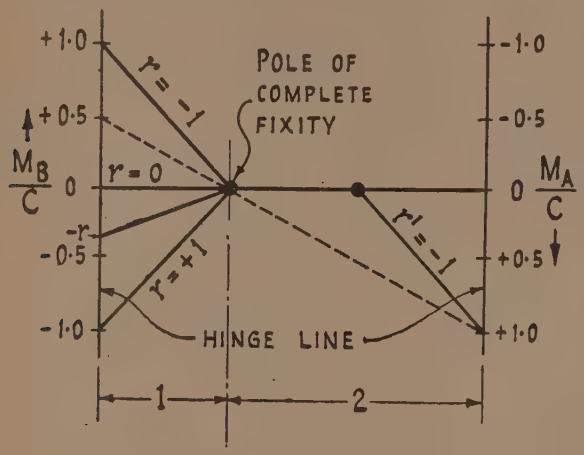
Where D is the Flexural Rigidity of the slab

chion (it does not matter which end) where both r and r' vary between $+1$ and -1. If there were a hinge it is clear that on releasing rC (i.e., " balancing ") there would be an applied moment on the stanchion of $-rC$, irrespective of the conditions at the other end. So that in Fig. 5(a) the " hinge line," which is a special case of a general degree of fixity, corresponds to the vertical line

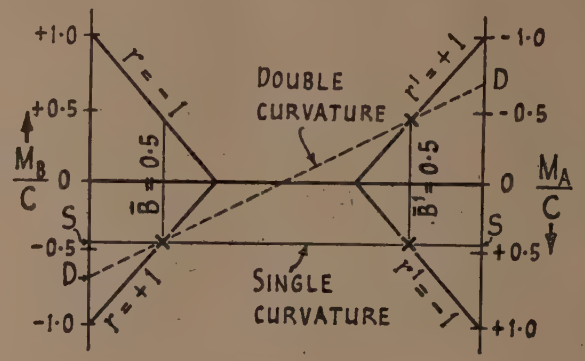
on which the stanchion moment $\frac{M_B}{C}$ is read. That is to say radial lines representing r intersect this line at $-r$.

We next arrange the radial lines r to pass through a "pole of complete fixity" situated one-third the distance along the diagram. Now consider a fixed end moment on the beams at end A equal to $-C$, ($r' = -1$). If end A were hinged this would give rise to a clockwise stanchion moment of $+C$ which would carry over $+\frac{1}{2}C$ to end B *providing end B were fixed.* This is quite clearly achieved in the diagram by the dotted line passing through the fixed pole and intersecting BB at $+\frac{1}{2}$.

With reference to Horne's original nomograms we may reasonably expect all other degree of fixity lines will be noticed that the principle of super-position of loads is clearly demonstrated for any given frame. Nomogram No. 1 is the completed diagram for the case of zero direct load, the axis having been made vertical to represent a stanchion.



(a)



(b)

Fig. 5.—Construction of nomogram at zero load

The Instability Problem

The problem of elastic instability under a direct load P introduces the important quantity αL, where

$$\alpha L = \sqrt{\frac{P}{EI}} \cdot L \quad \ldots\ldots \quad (1)$$

$$\text{Alternatively } \alpha L = \sqrt{\frac{f_s}{E}} \cdot \left(\frac{L}{k}\right) \quad \ldots\ldots \quad (2)$$

at each end of the stanchion to be parallel to the hinge lines, i.e., they become simply vertical lines, but their horizontal position has to be determined. The degree of fixity at either end* is expressed by the quantity

$$\bar{B} = \frac{K_c}{K_c+K_1+K_2+K_3} = \frac{K_c}{K_c+\Sigma K} = \frac{\text{Column stiffness}}{\text{Total stiffness at joint}}$$

Thus $\bar{B} = 1.0$ corresponds to a hinge, and $\bar{B} = 0$ corresponds to complete fixity. The easiest way of finding the location of these lines is to consider symmetrical single curvature (S/C) $r = +1$, $r' = -1$, Fig. 5b, and for any value of $\bar{B}$ (say 0.5) the value of $M_B = -M_A$ can easily be calculated. Notice that the scale at end A is reversed in sign to that at end B (when moments are measured clockwise), so that the moment line SS is actually the bending moment diagram measured from the base 00. Similarly when $r = +1$, $r' = +1$ the line DD clearly becomes the bending moment diagram in "pure" double curvature. This construction is found to hold for any values of $\bar{B}$ or r, whether symmetrical or unsymmetrical. The rule is to take the intersection of $\bar{B}$ and r at one end and to draw a line through this point and the intersection of $\bar{B}'$ and r' at the other end. This line is then the bending moment diagram. Incidentally this diagram may also be used for beam moments by feeding in the correct information. It

*Horne did not use this parameter for beams. The reason for the choice of this nomenclature is explained in reference (1).

where $f_s = \frac{P}{A}$ = average direct stress in the column.

k = radius of gyration about the appropriate axis.

For steel $E = 13{,}000$ tons/in.^{2} approx. so that

$$\alpha L = \frac{\sqrt{f_s}}{114} \cdot \frac{(L)}{(k)} \quad \ldots\ldots \quad (3)$$

which is a very convenient form since the designer normally calculates $\frac{L}{k}$ as a matter of course. Note, however, that in all these nomograms L is the *actual* length. The effective length is automatically taken care of.

It is convenient to adopt the functions X, Y, where

$$X = \frac{f}{4\varnothing^2 - f^2} \qquad Y = \frac{\varnothing}{4\varnothing^2 - f^2}$$

$$\text{and } f = 2\frac{[\alpha L \,.\, cosec\ \alpha L - 1]}{(\alpha L)^2}, \quad \varnothing = \frac{[1 - \alpha L\ cot\ \alpha L]}{(\alpha L)^2}$$

It can be shown that, in the absence of sway, the moments at the ends of the stanchion length AB, Fig. 2, are given by

$$M_B = -C_B \left[Y \cdot \frac{\Sigma K_A}{K_c} + Y^2 - \frac{X^2}{4} \right] - C_A \cdot \frac{X}{2} \cdot \frac{\Sigma K_B}{K_c} \Big/ \left(\frac{\Sigma K_A}{K_c} + Y \right)\left(\frac{\Sigma K_B}{K_c} + Y \right) - \frac{X^2}{4} \quad (4)$$

$$M_A = -C_A \left[Y \cdot \frac{\Sigma K_B}{K_c} + Y^2 - \frac{X^2}{4} \right] - C_B \cdot \frac{X}{2} \cdot \frac{\Sigma K_A}{K_c} \Big/ \left(\frac{\Sigma K_A}{K_c} + Y \right)\left(\frac{\Sigma K_B}{K_c} + Y \right) - \frac{X^2}{4} \quad (5)$$

where M_A and M_B act clockwise on the column, and C_A, C_B are clockwise out-of-balance F.E.M.'s at the joints as previously explained.

ΣK_A and ΣK_B are the total effective beam stiffnesses at the joints, which may include the adjacent column length as a first approximation. At high loads the stiffness of this adjacent column length may have to be modified somewhat as will be shown later.

The column will obviously buckle when the denominator of (4) and (5) becomes zero, as has been determined previously by other writers. It will be noticed that the buckling load depends only on the elastic restraints and the column stiffness as modified by the direct load expressed as (αL).

Expressed in such a manner there seems little hope of extending Horne's nomograms to cover elastic instability,

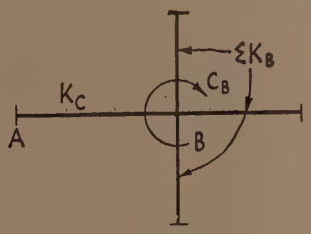


Fig. 6

but the key to the problem is seen when we revert to "degree of restraint" methods[5 6].

Thus, if we consider Fig. 6, then the "carry over" for a beam is

$$M_A = -C_B \cdot \frac{K_c}{K_c + \Sigma K_B} \cdot \frac{1}{2} \quad \ldots\ldots\ldots (6)$$

and it is known[6] that the modified stiffness at end A due to the degree of restraint at B is

$$K'_c = K_c \left[1 - \frac{K_c}{Kc + \Sigma K_B} \cdot \frac{1}{4} \right] \quad \ldots (7)$$

which may be looked upon as a "stiffness carry over." When direct load is present the author has found that the moment carry-over becomes

$$M_A = -C_B \cdot \frac{K_c Y}{K_c Y + \Sigma K_B} \cdot \left(\frac{X}{2Y} \right) \quad \ldots\ldots (6a)$$

and the modified stiffness

$$K'_c = K_c Y \left[1 - \frac{K_c Y}{K_c Y + \Sigma K_B} \cdot \left(\frac{X}{2Y} \right)^2 \right] \quad . (7a)$$

These results are of considerable interest, for in the general case the stiffness K_c is replaced by the reduced stiffness $K_c Y$ and the moment and stiffness carry-over factors become respectively $\left(\frac{X}{2Y} \right)$ and $\left(\frac{X}{2Y} \right)^2$.

When there is no direct load, $\alpha L = 0$, we find that $X = Y = 1.00$, and the carry-over factors become $\frac{1}{2}$ and $\frac{1}{4}$. With this transformation it is seen that algebraic operations are similar for moment distribution and for stiffness modification at all direct loads, including of course zero load.

We may therefore expect a nomogram for cases where instability will be present to be composed likewise of straight lines. The "pole of complete fixity" (Fig. 7)

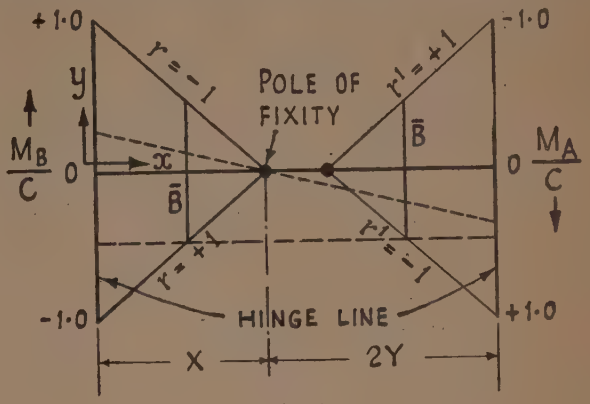


Fig. 7

is so arranged that when there is a hinge at the opposite end the applied moment becomes $-r'C$ and the moment carried over is $-r'C \cdot \frac{X}{2Y}$, since $\Sigma K_B = 0$ (equation (6a)).

It remains to fill in the vertical (degree of restraint) lines, which is carried out as before by solving the special cases of symmetrical single curvature. From equation (4) we obtain

$$\frac{M_B}{C} = \frac{-r}{\dfrac{\Sigma K}{K_c(Y - X/2)} + 1} \quad \text{for symmetrical single curvature} \quad (8)$$

or

$$\frac{M_B}{C} = -\frac{-r}{\left(\dfrac{1}{\bar{B}} - 1 \right) \dfrac{1}{Y - \dfrac{X}{2}} + 1} \quad \ldots\ldots\ldots (9)$$

from which we can plot vertical lines by inspection for any degree of restraint $\bar{B}$. This leads to a series of completed nomograms which are given for $\alpha L = 0$, 2, $2\frac{1}{2}$, $3\frac{1}{2}$, 4, $4\frac{1}{2}$, 5, from which the restraint moments can be obtained at either end no matter how assymmetrical be the frame or the loading.

Worked Example

Consider a stanchion length where the elastic restraints at the ends are represented by $\bar{B}$ .3, $= \bar{B}' =$ .20.

In using these nomograms it is important to multiply all direct loads and beam out-of-balance fixed end moments by the desired load factor first, and then to take the effective yield stress as the criterion (say 18 tons per sq. in. to agree with B.S.449). With the factored load the value of αL is easily determined graphically from the upper half of Fig. 8. Let us

suppose that $\alpha L = 2$. Referring to the appropriate nomogram (No. 2), then if the beam loads to be studied give rise to Double Curvature with equal out-of-balance moments we consider the radial lines $r = +1$, $r' = +1$. Find the intersection point of $r = +1$ with $\bar{B} = .3$; also the intersection point of $r' = +1$ and $\bar{B}' = .20$. Draw a line through these points (or stretch a length of cotton on a nomogram constructed to a large scale) and

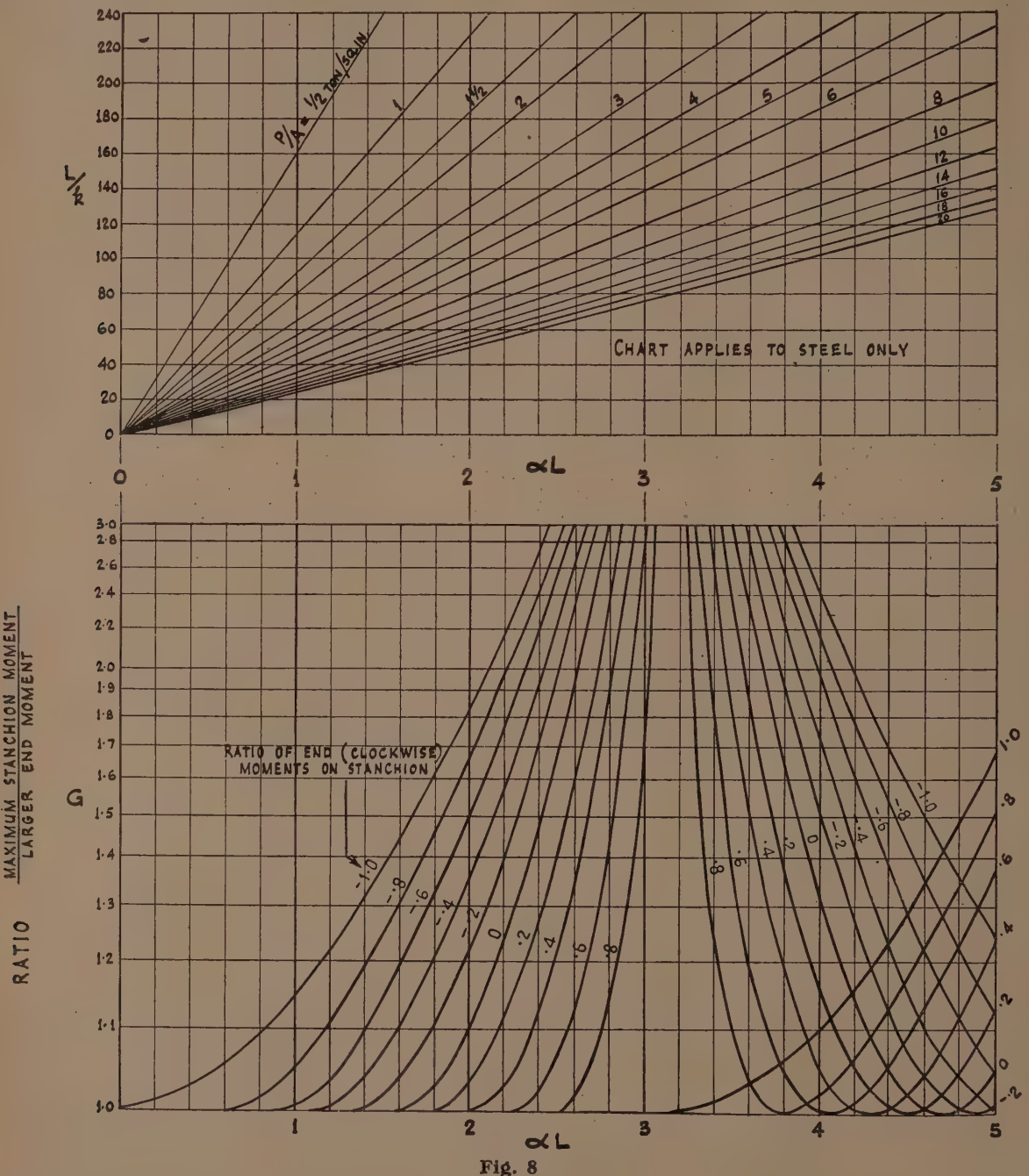


Fig. 8

the end moments are straightway read off the scales as −0.34 and −0.30 respectively (in terms of unit out-of-balance moments of +1). It will be shown later that the end moment −.34 is the maximum on the stanchion length at this direct load, and it will be seen in this case that it occurs at the end with the smaller degree of restraint.

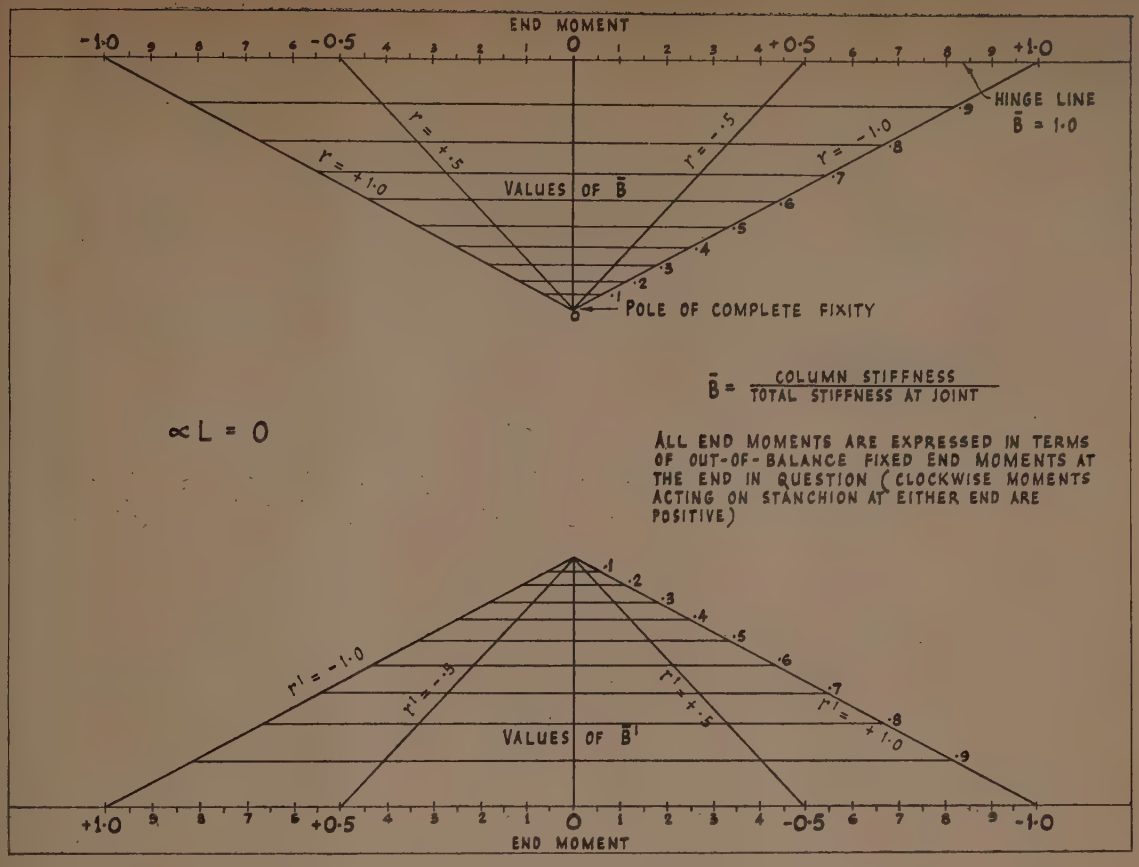
When αL exceeds π the radial lines now diverge or fan out in the opposite direction *but the corresponding end moment is still read on the same horizontal line as at the lower loads.* For instance when $\alpha L = 4$ the above example then gives the end moments as −.23 and −.30 respectively, that is to say the end moments hav now changed round in order of magnitude.

Formal Proof of the Nomograms, and Genera Rules for their Construction

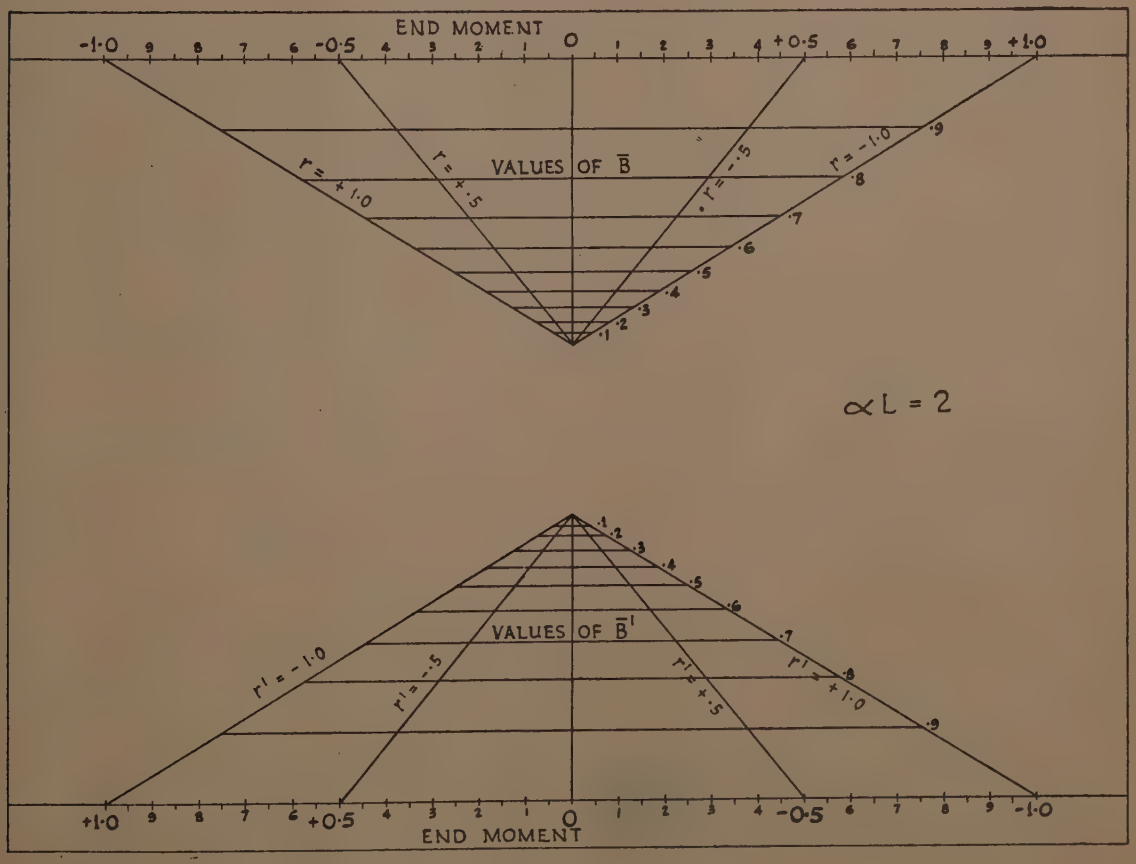
Most designers will no doubt accept the foregoin arguments as they stand, but the mathematically minded reader will observe that no strict formal proo has yet been given that the nomograms are indeed composed of nothing but straight lines.

Consider the radial r-lines of Fig. 7. Taking the co-ordinate axes x, y, as shown these lines all pass though the pole $\{X, 0\}$ and have a gradient of $\frac{+r}{X}$. Conse-quently the equation of a typical line is $y = \frac{r.x}{X} - r$ . . (10)

The vertical $\bar{B}$ lines must intersect the r lines at an ordinate given by



NOMOGRAM 1. ZERO DIRECT LOAD ∝ L = 0



NOMOGRAM 2

$$\frac{-r}{\left(\frac{1}{\bar{B}}-1\right)\left(\frac{1}{Y-\frac{X}{2}}\right)+1} \quad \text{from equation (9)}$$

The intersection point (x_1, y_1) of any $\bar{B}$ and r line can then be shown to be

Typical intersection point of first half of Nomogram

$$x_1 = \frac{X\left(\frac{1}{\bar{B}}-1\right)}{\left(\frac{1}{\bar{B}}-1\right)+\left(Y-\frac{X}{2}\right)} \quad . \quad (11)$$

$$y_1 = \frac{-r}{\left(\frac{1}{\bar{B}}-1\right)\left(\frac{1}{Y-\frac{X}{2}}\right)+1} \quad . \quad (12)$$

In like manner the second intersection point (x_2, y_2) of r' and $\bar{B}'$ lines in the other half of the nomogram can be found to be

$$x_2 = X+2Y-\frac{X\left(\frac{1}{\bar{B}'}-1\right)}{\left(\frac{1}{\bar{B}'}-1\right)+\left(Y-\frac{X}{2}\right)} \quad . \quad (13)$$

$$y_2 = +\frac{r'}{\left(\frac{1}{\bar{B}}-1\right)\left(\frac{1}{Y-\frac{X}{2}}\right)+1} \quad \text{by virtue of anti-symmetry} \quad (14)$$

The equation of a line passing through these two points is $\frac{y-y_1}{x-x_1} = \frac{y_2-y_1}{x_2-x_1}$, and the intercept y_o at $x = o$ is the required moment, i.e., $y_o =$

$$\frac{-x_1 y_2+y_1 x_2}{x_2-x_1} \quad . \quad (15)$$

Making use of (11), (12), (13) and (14) we find that after considerable manipulation the denominator of (15) reduces to

$$\frac{2Y-X}{\left(\frac{\Sigma K_A}{K_c}+Y-\frac{X}{2}\right)\left(\frac{\Sigma K_B}{K_c}+Y-\frac{X}{2}\right)} \times \left[\left(\frac{\Sigma K_A}{K_c}+Y\right)\left(\frac{\Sigma K_B}{K_c}+Y\right)-\frac{X^2}{4}\right]$$

whilst the numerator of (15) reduces to

$$\frac{2Y-X}{\left(\frac{\Sigma K_A}{K_c}+Y-\frac{X}{2}\right)\left(\frac{\Sigma K_B}{K_c}+Y-\frac{X}{2}\right)} \times \left[-r\left(Y.\frac{\Sigma K_A}{K_c}+Y^2-\frac{X^2}{4}\right)-r'\frac{X}{2}\frac{\Sigma K_B}{K_c}\right]$$

from which by definition

$$\frac{M_B}{C} = \frac{-r\left\{Y.\frac{\Sigma K_A}{K_c}+Y^2-\frac{X^2}{4}\right\}-r'.\frac{X}{2}.\frac{\Sigma K_B}{K_c}}{\left(\frac{\Sigma K_A}{K_c}+Y\right)\left(\frac{\Sigma K_B}{K_c}+Y\right)-\frac{X^2}{4}} \quad . \quad (17)$$

which coincides identically with the known value given in equation (4).

The moment at the opposite end obviously follows from symmetry of the diagram. Hence we have formally shown that the nomograms must consist of straight lines.

The general rule for constructing any nomogram is therefore to erect the out-of-balance Fixed End Moment (r) lines by equation (10) and the vertical lines by equations (11) or (12) whichever is most convenient for plotting.

Discussion of the Behaviour of the Stanchion

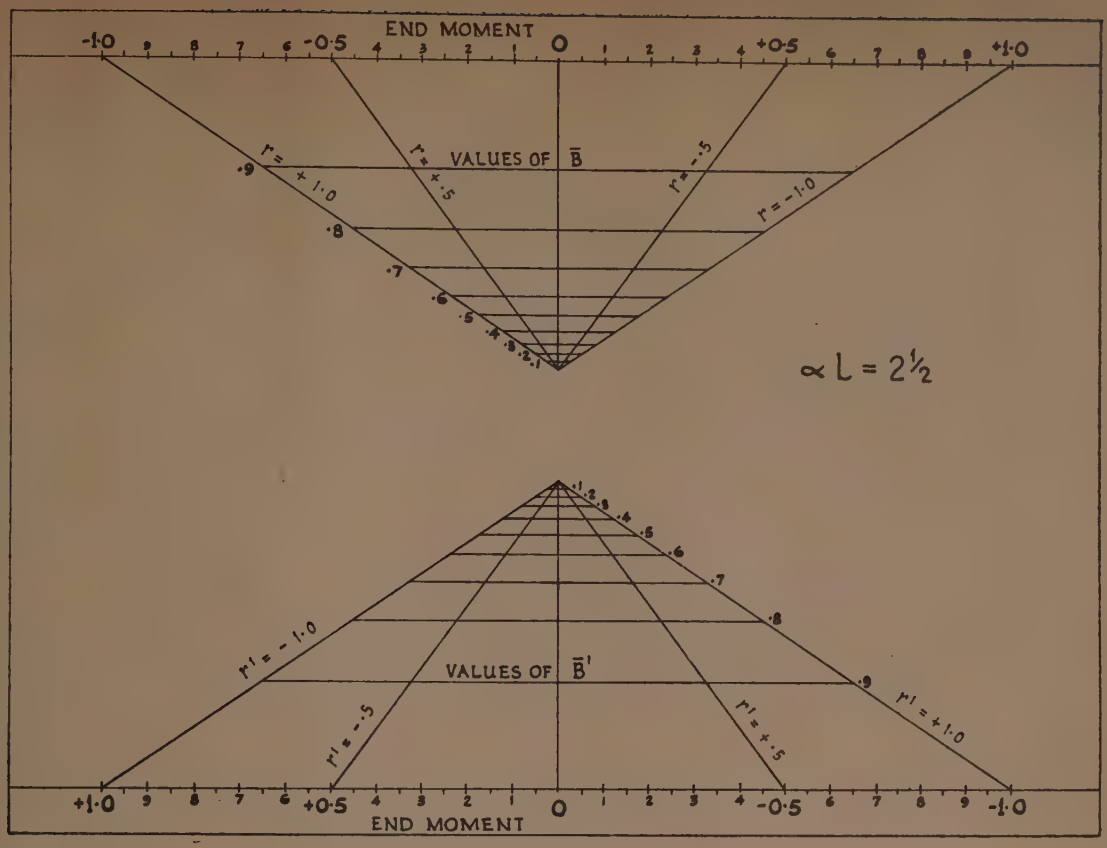
Apart from the special form in which the modified stiffness (7a) is expressed it is not claimed that any new principles are depicted by the nomograms, but that they show pictorially and almost instantaneously features which are all too easily overlooked. The most striking feature is that the nomogram for $\alpha L = \pi = 3.142$, corresponding to the Euler Load $\frac{\pi^2 E I}{L^2}$ shrinks to a single point since it can then be shown that $X = 2Y = \frac{\pi^2}{8}$, and from equation (11) all the elastic restraint lines ($\bar{B}$) congregate at a point exactly in the middle of the diagram. By drawing a series of lines through this point it is therefore clear that, in the absence of initial curvature, the restraining moments *must always be in a state of "pure" Double Curvature at the Euler Load, no matter how unsymmetrical the frame and loading may be.* In other words a state of "pure" Double Curvature is the only possible way of getting past $\alpha L = \pi$, as far as the stanchion length is concerned. It follows immediately that if a symmetrical frame is loaded in "pure" *single* curvature, and it must by symmetry remain in single curvature, the only way of satisfying this paradox is that the end moments both become zero, which may equally well be S/C or D/C (the zero vertical line of the nomogram).

The nomogram of course fails to give the actual values of the end moments at "π" but it can be shown by proceeding to limits that at the Euler Load

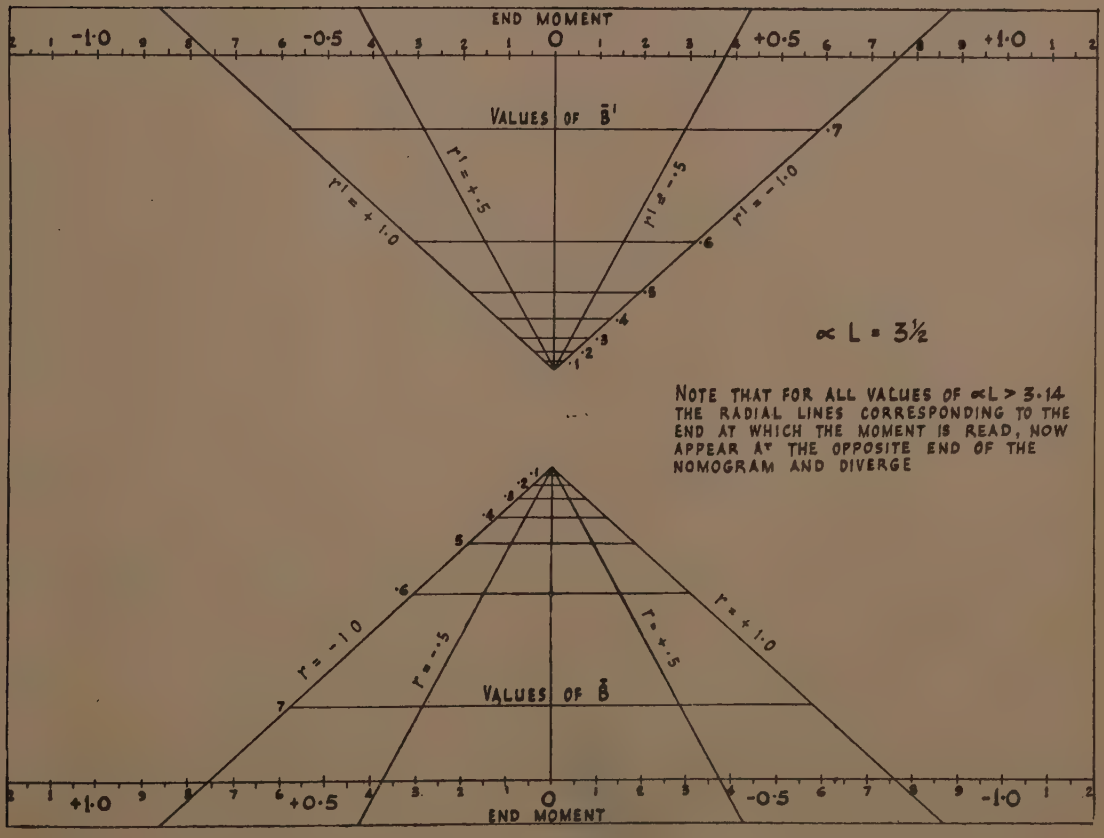
$$M_A = M_B = \frac{-C_A \cdot \frac{K_c}{\Sigma K_A} - C_B \cdot \frac{K_c}{\Sigma K_B}}{\left(\frac{4}{\pi}\right)^2+\frac{K_c}{\Sigma K_A}+\frac{Kc}{\Sigma K_B}} \quad . \quad (18)$$

being clockwise moments acting on the stanchion length. This is in general "pure" Double Curvature, only vanishing with a symmetrical frame when $C_A = -C_B$ (Single Curvature loading). If initial curvature is present there is a slight departure from pure D/C, but the ratio of the end moments is still unique.

This state of pure Double Curvature is thus uniquely defined, and it follows automatically that to multiply



NOMOGRAM 3



NOMOGRAM 4.

the end moments determined by the usual slope deflection methods by a given load factor (thus preserving a random ratio of M_B/M_A, in general not equal to $+1$) will mean that in no circumstances can the stanchion length be stable when the Euler Load $\frac{\pi^2 E I}{L^2}$ is reached, that is if the stanchion is still elastic. To neglect the change of end moments is of course quite safe and this basis was adopted by the Steel Structures Research Committee, and followed by the author[1], but it leads to no possibility of making use of the beneficial effects of the equivalent length concept, as no load is possible greater than $\frac{\pi^2 E I}{L^2}$. By contrast B.S. 449 for structural steelwork allows a small eccentricity of 2 in. or so to be used in conjunction with an equivalent length of $0.7\ L$, although whether this is entirely rational is outside the scope of this paper.

When αL is close to the apparently indeterminate case of "π," the designer is recommended to use the Nomogram for $\alpha L = 3\frac{1}{2}$ or else to interpolate between $\alpha L = 2\frac{1}{2}$ and $3\frac{1}{2}$. At "π" however, the real shear on the stanchion can be shown to be

$$\left(\frac{F_B}{\alpha}\right) = -\left(\frac{F_A}{\alpha}\right) = -\frac{C_B}{\pi}\left\{\frac{2K_c}{\Sigma K_B}+\frac{\pi^2}{4}\cdot\frac{K_c}{\Sigma K_B}\cdot\frac{K_c}{\Sigma K_A}\right\} + \frac{C_A}{\pi}\left\{\frac{2K_c}{\Sigma K_A}+\frac{\pi^2}{4}\cdot\frac{K_c}{\Sigma K_A}\cdot\frac{K_c}{\Sigma K_B}\right\} \Big/ \left(\frac{4}{\pi}\right)^2+\frac{K_c}{\Sigma K_A}+\frac{K_c}{\Sigma K_B} \quad . \quad (18a)$$

and the maximum moment is

$$M_{max} = \sqrt{M_B^2+\left\{\frac{F_B}{\alpha}\right\}^2} \quad . \quad . \quad . \quad . \quad . \quad (18b)$$

so that it is in fact actually determinate. The proof of this is too lengthy to reproduce here.

For values of αL greater than π, the nomogram, having shrunk to a point, reappears on the opposite side by virtue of equation (12). It is quite clear that with a symmetrical frame in single curvature the end moments, which were previously "acting" moments have now changed sign and have become restraining moments, and the configuration now rapidly becomes similar to the type noticed in plastic collapse. In using the nomogram, say for $\alpha L = 4$, it will be noticed that the lower radial lines now diverge continuously, although the corresponding moment is still read where the "moment line" intersects the upper scale.

The frame obviously buckles when the moments become infinity, i.e., when the horizontal $\bar{B}$ lines have receded to infinity. Thus with $\alpha L = 4$ a symmetrical frame would buckle when $\bar{B} = .68$ at both ends. It is easily verified from the Nomograms that this will hold for all types of loading except symmetrical Double Curvature, and the interesting feature arises that "pure" D/C has a peculiar stabilising effect at high loads which it would be dangerous to make use of. Any initial curvature or any loading other than "pure" symmetrical D/C will cause the column to "unwrap," which has also been noticed in plastic design (4). This unwrapping can be traced by the use of the nomogram when at high loads one restraint moment gains rapidly on the other in all cases of non-symmetrical Double Curvature.

We have noticed that a frame with $\bar{B} = .68$ at both ends buckles at $\alpha L = 4$, but there are obviously unsymmetrical combinations which would buckle at the same direct load when say $\bar{B} > .68$ and $\bar{B}' < .68$. The remainder of the diagram will be found to have reappeared from infinity, so to speak, at the opposite end and is shown dotted. This gives rise to a point of considerable interest, for the two halves of the nomogram can now overlap, the full lines of one half being superimposed on the dotted lines of the other half. When any two horizontal lines (or interpolations) belonging to opposite ends of the stanchion coincide, the moment line joining the two points will become horizontal no matter what the beam Fixed End Moments are. Thus buckling combinations can be picked out instantaneously, for example at $\alpha L = 4$ we have $\{\bar{B} = .9, \bar{B}' = .47\}$ and $\{\bar{B} = .85, \bar{B}' = .52\}$.

It should be pointed out that such high values of $\bar{B}$ are most unlikely to occur in practice, the practical limit for stanchions being about 0.4, so that not all parts of the complete nomogram may be required. Nevertheless the situation changes when we come to a study of continuous stanchions.

Procedure for Using Nomograms

An auxiliary chart, Fig. 8, which is self-explanatory, is given for the graphical determination of αL. The end moments are not of course necessarily the maximum moments, and Fig. 8 also gives the ratio of the maximum moment to the maximum end moment (regardless of sign). This is expressed as an enhancement factor G which is always greater than unity. Obviously G depends on the ratio of end moments. The rule is to choose the largest (arithmetical) end moment, say M_A, and to calculate $\frac{M_B}{M_A}$ treating both as clockwise positive (Hardy Cross Convention), taking care of sign, and then read off the enhancement factor G. In many cases the designer will find $G = 1.000$ indicating that the maximum moment is actually at the end. Professor Winter[10] has produced a similar chart, which did not however extend beyond "π." In this extended region it will be seen that there are two branches to the curves, and both branches are valid. There is however only one result for G and in practice it is not usual to find much deviation from unity.

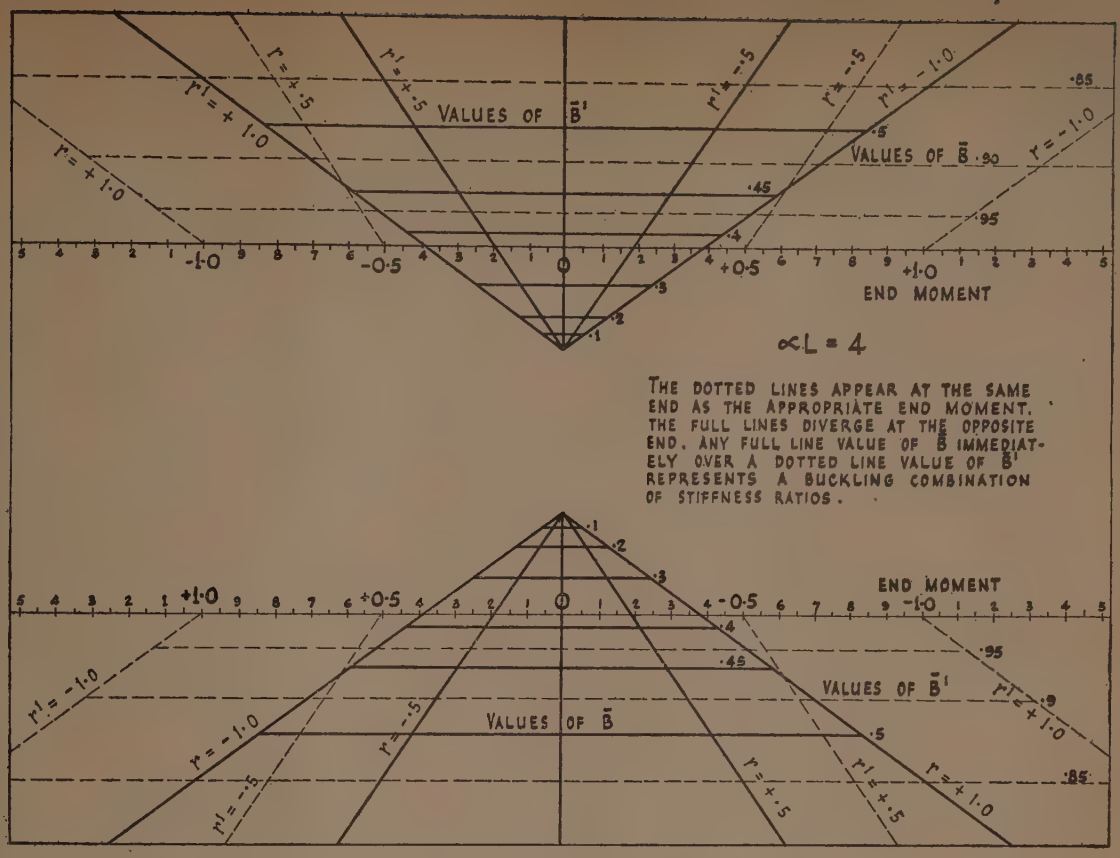
Example

Table I may serve as an exercise in the use of the nomograms and demonstrates the phenomenon of "unwrapping."

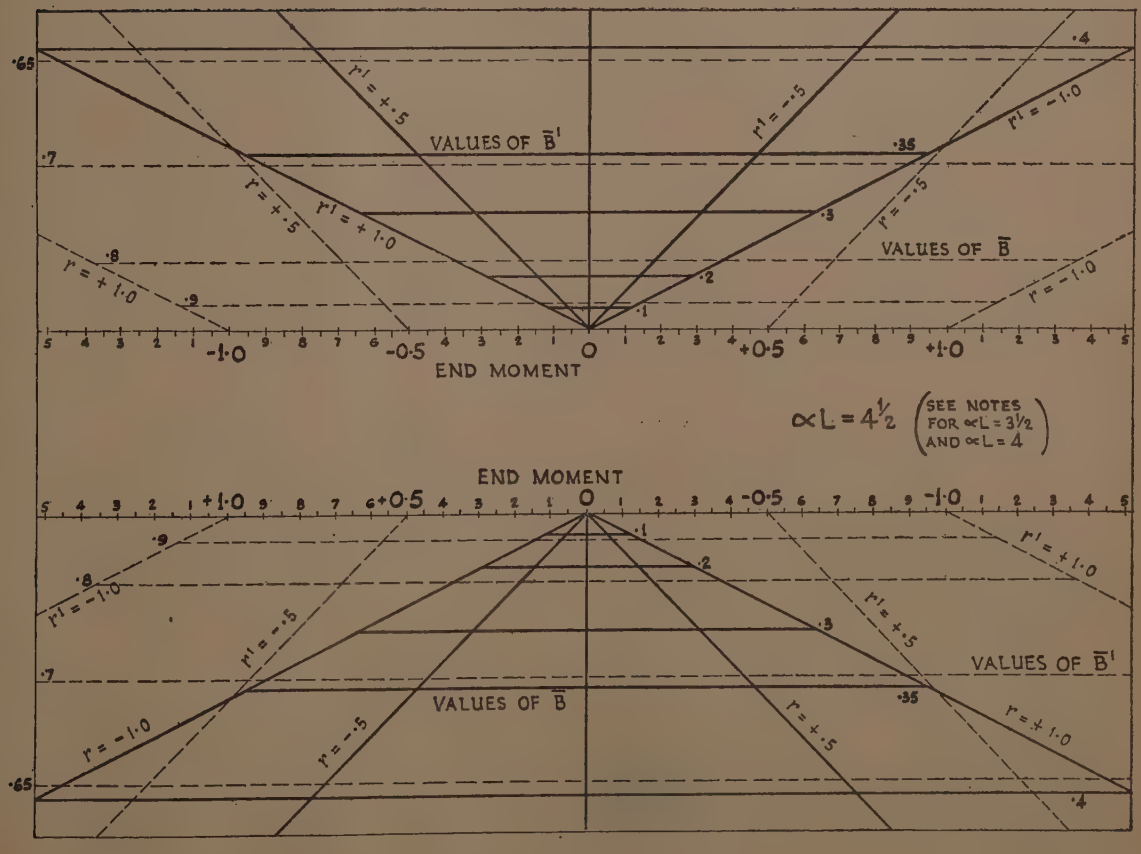
TABLE I.—Solution of Unsymmetrical Frame in Double Curvature:

$r = +1,\ r' = +1$

αL	End Moments		G (Fig. 8)	M_{max}
	$B = .05$	$B' = .20$		
0	—.14	—.22	1	.22
2	—.15	—.21	1	.21
4	—.18	—.09	1.03	.185
5	—.31	+.09	1.01	.315


VALUES OF $\bar{B}'$
VALUES OF $\bar{B}$
END MOMENT
$\propto L = 4$
THE DOTTED LINES APPEAR AT THE SAME END AS THE APPROPRIATE END MOMENT. THE FULL LINES DIVERGE AT THE OPPOSITE END. ANY FULL LINE VALUE OF $\bar{B}$ IMMEDIATELY OVER A DOTTED LINE VALUE OF $\bar{B}'$ REPRESENTS A BUCKLING COMBINATION OF STIFFNESS RATIOS.
VALUES OF $\bar{B}'$
VALUES OF $\bar{B}$
END MOMENT

NOMOGRAM 5


VALUES OF $\bar{B}'$
VALUES OF $\bar{B}$
END MOMENT
$\propto L = 4\frac{1}{2}$ (SEE NOTES FOR $\propto L = 3\frac{1}{2}$ AND $\propto L = 4$)
END MOMENT
VALUES OF $\bar{B}'$
VALUES OF $\bar{B}$

NOMOGRAM 6

The progressive states of the stanchion are shown pictorially in Fig. 10. It is of considerable interest to note that whereas the weaker ($\bar{B}' = .20$) end gives rise to the largest moment at small loads, this moment is eventually lost altogether and changes sign. The opposite (strong) end gains all the time and the stanchion eventually unwraps about the strong end in the characteristic pattern. The additional effects which plasticity would introduce will not be discussed here. It suffices to note that until "unwrapping" sets in Double Curvature actually gives rise to an alleviating effect (*vide* the last column).

Continuous Stanchions

Strictly speaking the foregoing only applies to the conditions at the roof of a multi-storey building where the elastic restraint consists of beams only. The main purpose of this paper, however, is to present the principles of the application of this type of nomogram to problems involving combined bending and instability, and nomograms can be easily constructed to deal with various cases. It is not within the scope of the paper to discuss the multi-storey building in general. Nevertheless some indication must be given of how to deal with the problem of continuous stanchions.

Referring to the continuous stanchion of Fig. 2, then, if the extreme ends are encastered, all that it is necessary to do is to substitute $K_5 Y$ and $K^5 Y$, for the upper and lower stanchion lengths of stiffness K_2 and K_5 respectively. (Values of Y are given in the Appendix.) This is assuming that $\sqrt{\frac{P}{EI}}.L$ has the same value for adjacent stanchion lengths, which may not be strictly true, but the whole design would become virtually impossible if every variable were included. However, this simplification is too optimistic, for if we examine the worst possible arrangements of load for Double Curvature and Single Curvature (Fig. 9), as a first

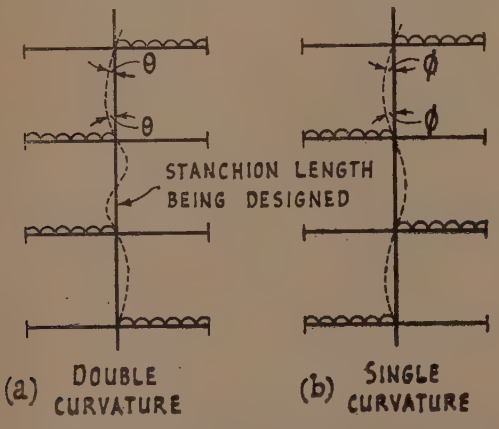


Fig. 9.—Worst live load arrangements for a continuous stanchion

approximation, it seems reasonable to assume the adjacent stanchion lengths to be in single curvature. This "induced stiffness" takes account of the moment carry-over automatically, and at zero direct load is tantamount to using $\frac{1}{2} K$ instead of K. This would no doubt be a pessimistic design, for it assumes a balance of dead loads which would otherwise induce double curvature and stiffen up the frame. It is tempting to assume a hinge instead (stiffness $\frac{3}{4} K$) but this has an adverse effect at high direct loads and would always lead to the conclusion that the whole frame would buckle at less than $\alpha L = 4.5$. This point is seen by the following Table of equivalent stiffnesses.

TABLE II.—Equivalent Stiffnesses of Upper and Lower Stanchion Lenghts

Direct Load aL	Fixed Ends $K' = KY$	Induced Single Curvature $K' = K\left(Y - \frac{X}{2}\right)$	Hinged $K' = K\left(Y - \frac{X}{4Y}\right)$
0	1	$\frac{1}{2}$	$\frac{3}{4}$
π (Euler)	.617	0	.117
4·49	0	—0.9	— ∞ (Buckled)
5	—.48	—1.67	Buckled

Negative stiffnesses occur when the column length is dragging the beams over with it prior to buckling. Buckling occurs when the sum total of stiffnesses, positive and negative, at a joint become zero. It will be

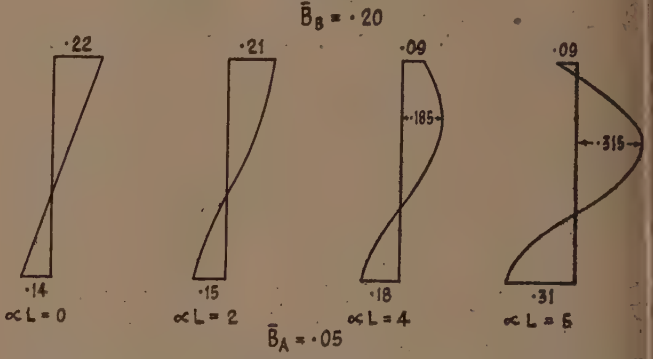


Fig. 10.—Bending moment diagrams for double curvature ($C_A/B_B = 1.0$) but with unsymmetrical frame ($\bar{B}_A = .05$; $\bar{B}_B = .20$)

seen that the concept of "induced single curvature" is conservative and moreover it preserves the elastic restraints at the storey level above, which a hinge does not.

This being the case, we will assume that the stanchion stiffness K_c is repeated in the storey above*, and is replaced by an equivalent stiffness $K_c \left(Y - \frac{X}{2}\right)$.

Consequently the nominal value of $\bar{B}$ is $\frac{K_c}{K_c + K_c + \Sigma K_{\text{beams}}}$ for design purposes, whereas the frame behaves as if the equivalent value is

$$\bar{B}_e = \frac{K_c}{K_c + K_c\left(Y - \frac{X}{2}\right) + \Sigma K_{\text{beams}}}$$

from which it is easy to show that for internal stanchions

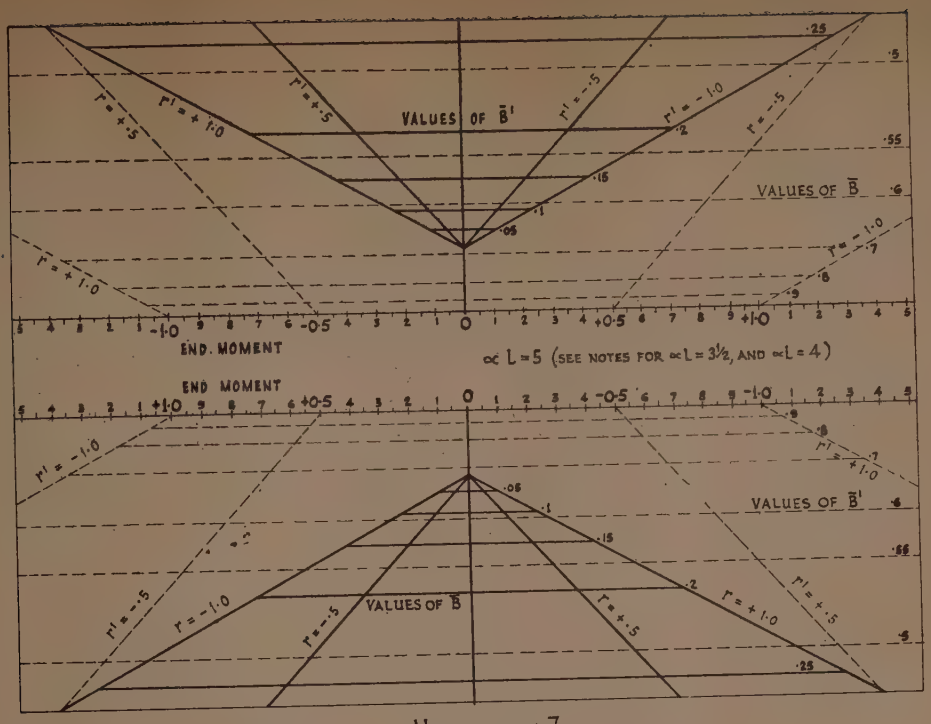
$$\frac{1}{\bar{B}_e} = \frac{1}{\bar{B}} - 1 + Y - \frac{X}{2} \quad \ldots\ldots\ldots \quad (19)$$

For external stanchions the adjacent lengths are bent into double curvature, but not as much as symmetrical double curvature. Consequently it would be conservative to take the adjacent lengths with encastered ends, in which case

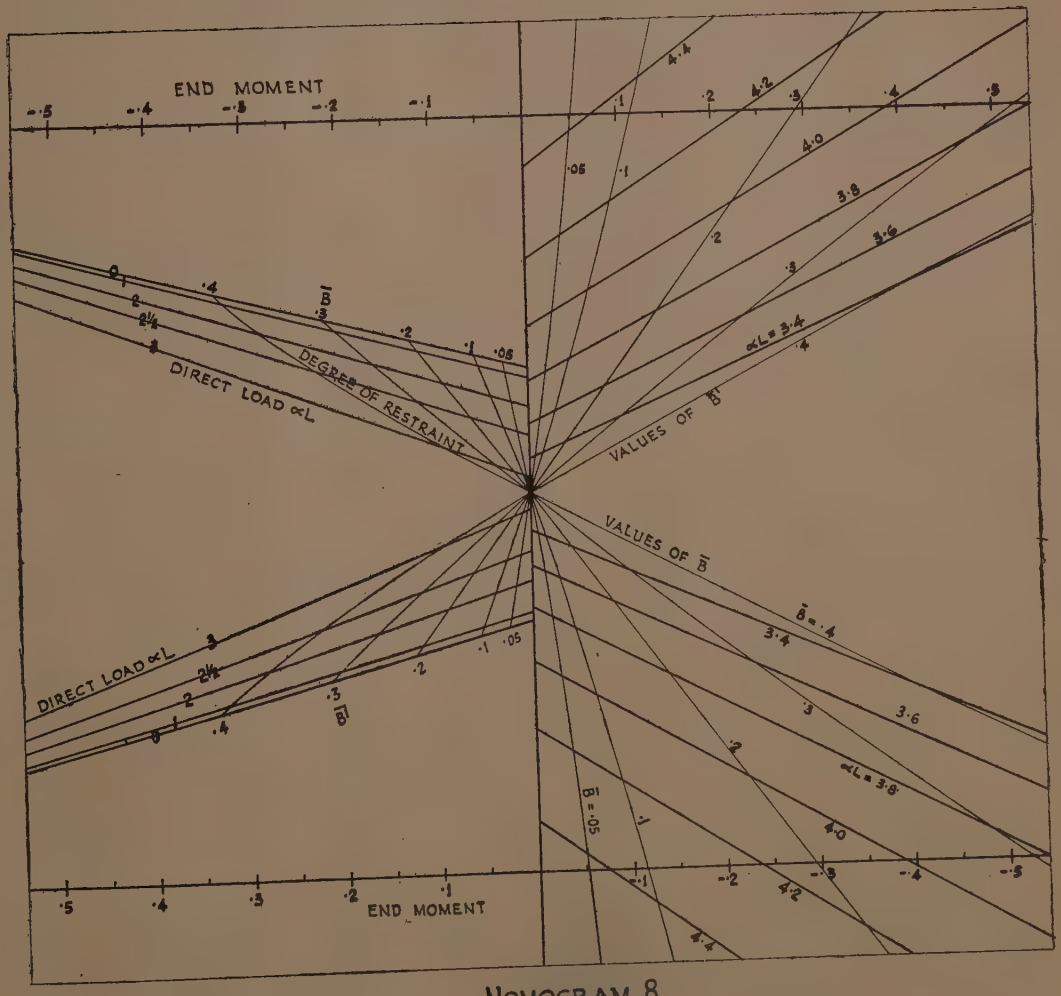
$$\frac{1}{\bar{B}_e} = \frac{1}{\bar{B}} - 1 + Y \text{ for external stanchions} \quad (19a)$$

All that it is necessary to do therefore is to *shift the horizontal (elastic restraint) lines of the nomogram upwards or downwards according to the above rules*, and the behaviour of a continuous stanchion length will be

*It must be remembered that if there is a stanchion taper, the direct load is also "tapered."


VALUES OF $\bar{B}'$
VALUES OF $\bar{B}$
END MOMENT
$\propto L = 5$ (SEE NOTES FOR $\propto L = 3\frac{1}{2}$, AND $\propto L = 4$)
END MOMENT
VALUES OF $\bar{B}'$
VALUES OF $\bar{B}$

NOMOGRAM 7



NOMOGRAM 8

SINGLE CURVATURE LOADING WITH UNSYMMETRICAL FRAMES
GENERAL CHART FOR CONTINUOUS STANCHIONS: ALL DIRECT LOADS

predicted. The following table gives some typical numerical values :—

TABLE III.—Equivalent Elastic Restraint for Continuous Internal Stanchions

Nominal $\frac{B}{}$	$\bar{B}e$ = Equivalent $\bar{B}$ for continuous stanchions						Values of aL
	0	1	2	3	4	5	
.05	.0512	.0513	.0517	.0524	.0539	.0577	
.1	.105	.106	.107	.110	.117	.137	
.15	.162	.163	.166	.175	.192	.251	
.2	.222	.224	.231	.247	.282	.430	
.25	.285	.289	.301	.327	.393	.753	
.3	.352	.358	.377	.419	.533	—	
.35	.424	.432	.458	.523	.714	—	
.4	.5	.510	.549	.643	.959	—	

The nomograms may be altered quite simply ; for example at $aL = 2$ the horizontal line which will still be labelled 0.3 is shifted to the position occupied by $\bar{B} = .377$ on the existing diagram.

Initial Curvature

Compared with the effects which can be produced by the beam loads themselves, the effects of initial curvature (i.e., crookedness) can be often quite incidental. Provided a state of "pure" Double Curvature is avoided for design purposes, the effects of initial curvature can be taken care of by erring on the side of induced Single Curvature. For example, the design of external stanchions according to $C_A/C_B = 0$, instead of some degree of Double Curvature, would almost certainly swamp the effects of initial curvature unless the beams were remarkably stiff. However, the author has found that if it is required to investigate the phenomenon of initial curvature all that it is necessary to do is to consider the fixed end moments, which accrue when a curved strut is encastered at both ends and subjected to direct load, as being the out-of-balance F.E.M. This must be done algebraically.

The F.E.M. of a curved encastered strut under load can be shown to be :—

$$C'_\varepsilon = \frac{2P_\varepsilon}{\pi} \cdot \frac{\pi^2}{\pi^2 - \alpha^2 L^2} \cdot (2Y - X) \quad . \quad . \quad (20)$$

where ε is the maximum out-of-straightness of sinusoidal shape and P is the direct load.

Thus we apply $+C'_\varepsilon$ at one end and $-C'_\varepsilon$ at the other end, and the resulting restraint moment line is clearly analogous to single curvature.

To the results read off the nomogram (which will be opposite in sign) we must add $+C'_\varepsilon$ and $-C'_\varepsilon$, which are of course still acting on the stanchion length.

Let the resulting moments at the ends of the stanchion length be $\pm M_\varepsilon$ for symmetrical frames. (For asymmetrical frames it is sufficient to take the mean degree of restraint at both ends.)

Then the maximum moment due to initial curvature is equal to

$$-M_\varepsilon \cdot \sec \frac{\alpha L}{2} + P_\varepsilon \cdot \frac{\pi^2}{\pi^2 - \alpha^2 L^2} \quad . \quad . \quad (21)$$

Future Design Methods Based on the Nomograms

The nomograms give a rapid solution of the behaviour of a stanchion in the elastic phase, and can be of immediate use for the designer who is well versed in the problems associated with multi-storey buildings. No attempt has been made of course to present a set of design rules, and further simplifications are being studied. A systematic analysis has already revealed that it is both unnecessary and undesirable to base an elastic method of design of stanchions on the Double Curvature condition, and hence the present indications are that, until a sufficiently simple, comprehensive collapse (or "ultimate") design is available, which includes all the effects of elastic instability, the stanchions might be designed on a basis of Single Curvature. It is true that external stanchions must be in some kind of double curvature, but here the worst case is likely to be when the out-of-balance fixed end moments are C and zero (being the greatest possible digression towards single curvature).

The complete design can then be embodied in a single chart for each case by extracting from the Nomograms the individual lines $r = +1\,r' + -1$, and $r = +1\,r' = 0$, respectively for internal and external stanchions. For example Nomogram 8 indicates the skeleton form of this comprehensive chart for internal stanchions, which is valid at all loads and slenderness ratios. The correction for continuity of stanchions (19) has been included in the chart.

One such chart would then suffice for the design of all continuous internal stanchions, and would automatically take into consideration the effects of lack of symmetry in the restraints.

Acknowledgments

The work described in this paper forms part of the programme of the Building Research Board and is published by permission of the Director of Building Research.

APPENDIX

Functions X and Y concerning buckling of Stanchions

L	X	Y
0	1.0	1.0
1	1.0172	0.9662
2	1.0761	0.8591
2.5	1.1286	0.7719
3	1.2057	0.6561
3.5	1.3212	0.5021
4.0	1.5018	0.2933
4.5	1.8070	—0.0048
5.0	2.3922	—0.4772

Further values can be obtained from reference 10.

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[1]Wood, R. H. "An Economical Design of Rigid Steel Frames for Multi-Storey Buildings." National Building Studies Research Paper No. 10. H.M.S.O. 1951.

[2]Final Report, Steel Structures Research Committee, H.M.S.O. 1936.

[3]Horne, M. R. "Maximum Beam Moments in Welded Building Frames." THE STRUCTURAL ENGINEER, May, 1950.

[4]Baker, J. F., and J. W. Roderick. "Investigation into the behaviour of Welded Rigid Frame Structures—Further Tests on Stanchions." 7th Interim Report. British Welding Research Association, London.

[5]Shepley, E. "Continuous Beam Structures." Concrete Publications, Ltd., 1942.

[6]Wood, R. H., and E. Goodwin. "Degree of Fixity Methods for certain Sway Problems." THE STRUCTURAL ENGINEER, July, 1952.

[7]Beaufoy, L. A., and A. F. S. Diwan. "Analysis of Continuous Structures by the Stiffness Factors Method." QU. JOURNAL APP. MECHANICS. Vol. II, Part 3, Sept., 1949.

[8]Wood, R. H. "Studies in Composite Construction : Part I : The Composite Action of Brick Panel Walls supported on Reinforced Concrete Beams." National Building Studies Research Paper No. 13. H.M.S.O., 1952.

[9]Wood, R. H. "Studies in Composite Construction, Part II." To be published. National Building Studies, H.M.S.O.

[10]Winter, Hsu, Koo and Lob. "Buckling of Trusses and Rigid Frames." Cornell Univ. Exp. Sta. Bull. No. 36. 1948.

Notes of Interest

Members of all grades are invited to submit "practice notes" of individual operations such as those described below, whose size and scope are not perhaps sufficient to justify a full-length article or paper, but which possess features of special or general interest to the Institution

Repairs to Roof Structure of the New Theatre, Northampton

By A. B. Grant, B.Sc.(Eng.),Lond., A.M.I.C.E., A.M.I.Struct.E.

Introduction

During an inspection of the roof over the stage at the New Theatre, Northampton, it was discovered that the structure had sagged considerably and urgent measures were considered necessary to reinstate it.

The New Theatre was built in 1910-1911, the layout of the stage roof being as shown in Fig. 2. Five roof trusses at approximately 9 ft. centres span 40 ft. from the proscenium wall on to a timber truss which spans parallel to the rear wall.

This truss is supported on two side trusses which are approximately 60 ft. apart and span between the proscenium wall and the rear wall to the stage.

The reactions from the side-trusses are carried by brick piers built into the corners of the rear wall.

The roof covering consists of slates on close boarding. Attached to decking between the pitched roof trusses are a series of rows of pulleys required for manipulation of the curtains.

A detailed survey revealed that the roof superstructure had sagged by a maximum of 6 in., due to the main truss buckling 13 in. in the horizontal direction and hence deflecting vertically by up to 6 in. at midspan. Because of this movement, the load from the roof superstructure had been transferred to the rear wall through the mansard rafters, and the horizontal component of this thrust had caused the wall to crack and lean outwards by up to 9 in. at the top centre.

The general condition of the timber itself was good.

Original Scheme

A scheme was originally proposed for reinstatement by building a new steel lattice girder alongside the distorted timber truss, removing the latter, demolishing the upper portion of the walls and rebuilding to the original line and level. This scheme was however abandoned for the following reasons : (a) the cost was considered excessive, (b) it would necessitate regular maintenance, (c) the loads would have been transferred from the rear to side walls, and these were possibly not capable of sustaining them, and (d) as the theatre could only be closed for about six weeks, it was not considered possible to complete the work in time.

Procedure Adopted

It was therefore decided to proceed on the basis of restoring as far as possible the original arrangement ; thereby saving considerably in material, time and cost.

The roof superstructure was to be lifted to its correct level, thus relieving the rear wall and the main truss of any loading. The wall was then to be demolished to a level below the area of cracking, and the main truss restored to its correct position, replacing any defective material and providing suitable lateral bracing to prevent any further tendency to distort.

Jacking Operation

Scaffold towers were erected from the basement to the underside of each of the five roof trusses at the end nearer the main truss, mechanical jacks being inserted between the head of each tower and the underside of each roof truss (Fig. 3). Care was taken to ensure that the towers were placed plumb below the point of application of the jacks in order to avoid any eccentricity on the scaffolding. The whole structure was well braced together in two directions. The estimated load to be raised by each jack was of the order of two tons.

Repairs to Wall and Main Truss

In order to avoid dislodging slates and causing undue movement between the various members of the roof superstructure, it was decided to proceed from the centre roof truss outward. Thus the centre truss was first jacked up to the level of the adjacent two, and then these centre three trusses were jacked up simultaneously to the level of the outer two. Finally all five trusses were jacked up simultaneously until the whole roof superstructure was level. The total "lift" was approximately 6 in. The entire roof cladding was found to be still intact and no repairs were required.

By this means all loading was removed from the main truss and the rear wall. Separate jacks were used to support the two side trusses and free the rear wall of any loads from them.

It was now possible to commence demolishing the rear wall. To do this it was found necessary to support the roof and the first floor of the dressing-room annexe which spanned on to wall from behind the stage.

The wall was rebuilt in flettons, except for the end piers, for which engineering bricks were used in view of the load which had to be carried and the considerable length of unsupported work.

While the wall was being replaced, attention was turned to the main truss.

When the scheme for reinstatement was being prepared, the exact condition of the timber in the lower boom was not known, owing to the impossibility of making a close inspection. It was hoped in spite of the large sideways deflection of the member, that the timber was still in good condition and would be suitable for re-use. Metal brackets were manufactured and these were to be fixed to each side of the lower boom ; wire ropes were to be run between these brackets and attached

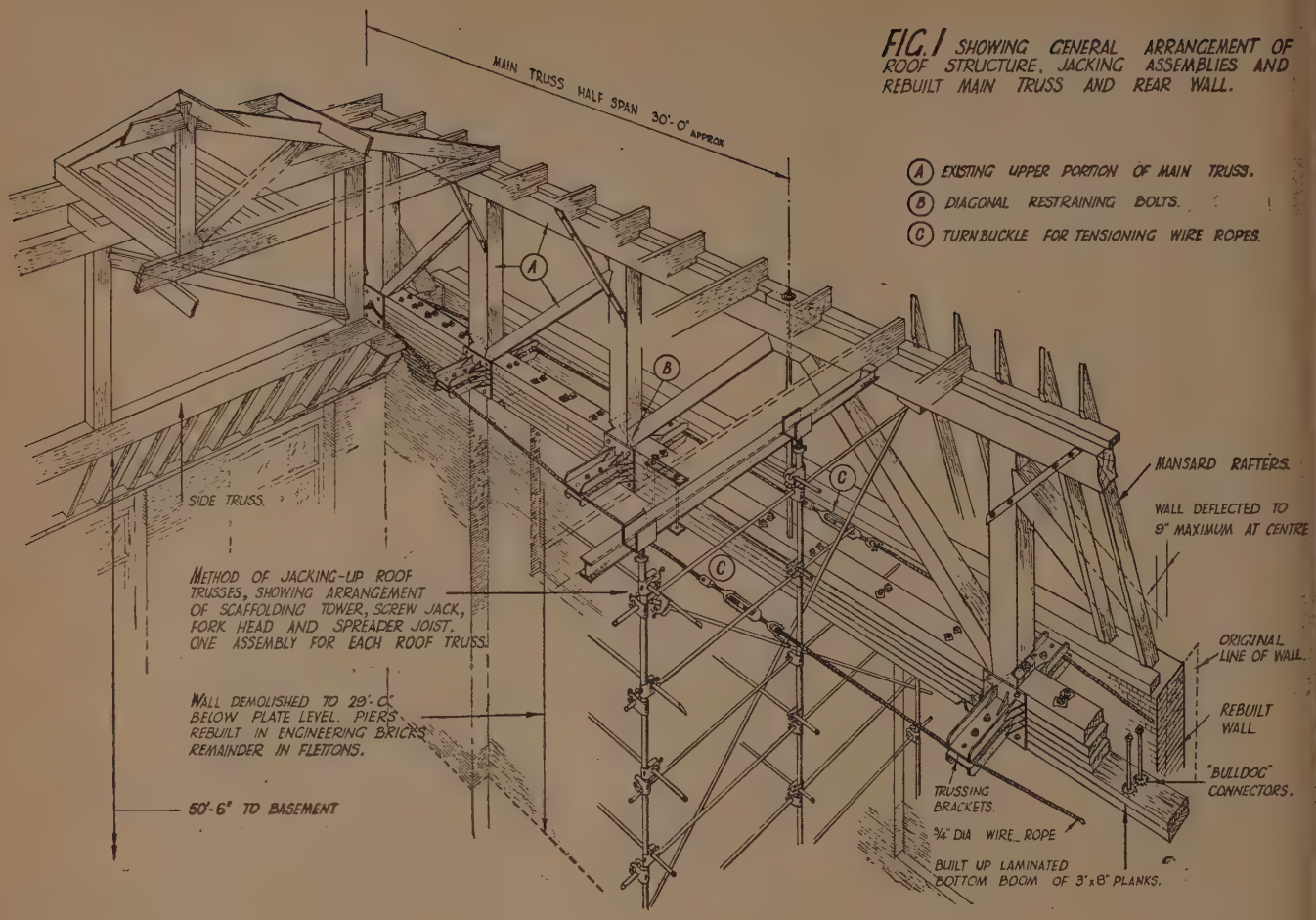

MAIN TRUSS HALF SPAN 30'-0" APPROX
A
B
C
C
SIDE TRUSS.
METHOD OF JACKING-UP ROOF TRUSSES, SHOWING ARRANGEMENT OF SCAFFOLDING TOWER, SCREW JACK, FORK HEAD AND SPREADER JOIST. ONE ASSEMBLY FOR EACH ROOF TRUSS.
WALL DEMOLISHED TO 29'-0" BELOW PLATE LEVEL. PIERS REBUILT IN ENGINEERING BRICKS REMAINDER IN FLETTONS.
50'-6" TO BASEMENT
MANSARD RAFTERS.
WALL DEFLECTED TO 9" MAXIMUM AT CENTRE
ORIGINAL LINE OF WALL.
REBUILT WALL
"BULLDOG" CONNECTORS.
TRUSSING BRACKETS.
¾" DIA WIRE ROPE
BUILT UP LAMINATED BOTTOM BOOM OF 3"x8" PLANKS.

FIG. 1 SHOWING GENERAL ARRANGEMENT OF ROOF STRUCTURE, JACKING ASSEMBLIES AND REBUILT MAIN TRUSS AND REAR WALL.

(A) EXISTING UPPER PORTION OF MAIN TRUSS.
(B) DIAGONAL RESTRAINING BOLTS.
(C) TURNBUCKLE FOR TENSIONING WIRE ROPES.

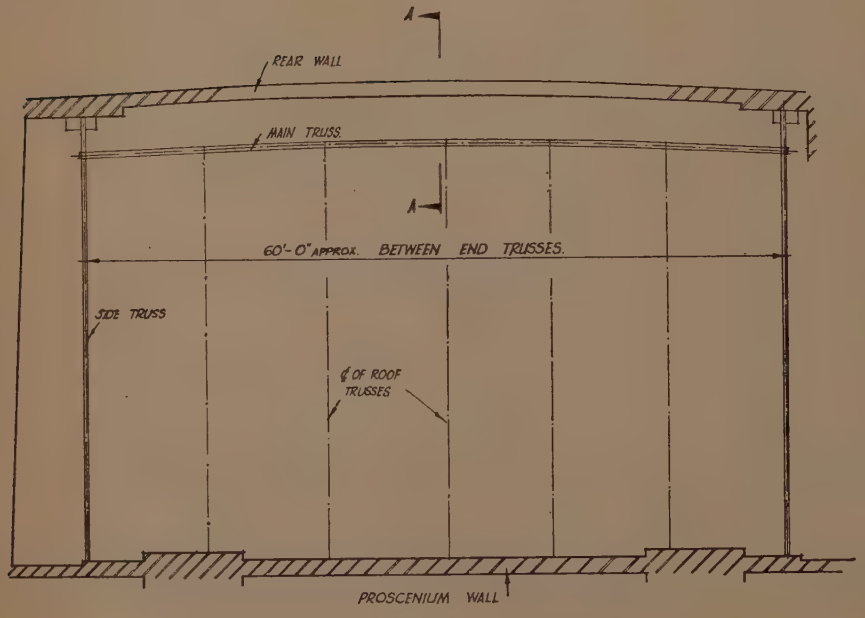
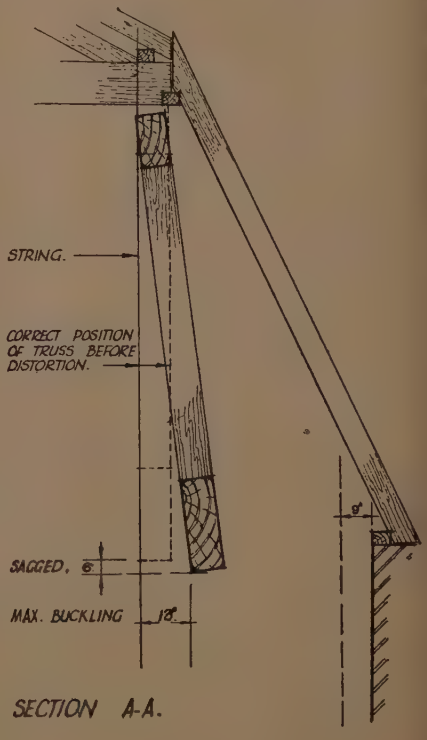


FIG. 2
REFLECTED PLAN OF STAGE ROOF STRUCTURE.



SECTION A-A.

at midspan to turnbuckles. By adjustment, tension could thus be applied to either or both ropes. The arrangement is shown in Fig. 1.

When the truss came to be examined during reconstruction, however, it was found that the timber in the lower boom had developed extensive splits and was not suitable for re-use. Moreover, owing to the large lateral movements, the various panel joints had opened up and required attention. The timber in top boom and in the diagonals and struts were all in good condition.

The original lower boom of the truss was made from a single piece of timber, 15 inches deep, 8 inches wide and approximately 58 feet long. It was obviously out of the question to replace by a similar member both from the view of obtaining and of fixing in position. It was

The mild steel trussing brackets were next fixed in position and the wire ropes and turnbuckles assembled. The whole truss was trued up before sufficient tension was applied to each wire rope through the turnbuckles to take up all the slack. It was now possible to remove the scaffolding brackets supporting the truss at its top panel points and allow it to carry its own load.

Fig. 1 shows the details of the repaired truss.

When the rear wall had been rebuilt to the correct level, it was possible to release the jacks holding up the roof structure. This was done in reverse order to the order of the jacking up, the last truss to be lowered back on to the main truss being the central one.

The maximum measured vertical deflection of the main truss after the entire roof load had been trans-

Fig. 3

therefore decided to build up the lower boom by laminating eight-inch wide by three-inch deep planks of Columbia pine. The planks were of random length, varying between fifteen and twenty-five feet.

The whole truss was first supported at the panel points of the top boom on brackets fixed to the scaffolding. The defective lower member was then cut out in sections and removed. This left the required working space for assembling the new lower boom.

Since the member had to be built up *in situ*, the planks were hoisted up and laid out on a staging immediately below their future position in the truss. The layers were connected together with " Bulldog " timber connectors, the positions of which were decided by a check calculation on the forces in the truss under full loading conditions.

When the new lower boom was complete it was then fixed to the main part of the truss. Diagonal bolts were used to connect posts and diagonals to the bottom boom, the size of these bolts being determined by the forces acting at the respective joint.

In repairing the joints at the top of the truss, diagonal straps were used in preference to diagonal bolts, owing to the impossibility of drilling the necessary holes immediately below the roof boarding.

ferred on to it was $\frac{1}{4}$ in. With the main truss loaded, part of the tension force in the lower boom was now carried by the wire ropes, and this tension gave the entire lower boom a high degree of rigidity in the lateral direction. This was considered of assistance in resisting any possible future buckling of the truss under dynamic loads due to raising stage scenery or curtains.

A further inspection was carried out after the stage had been in use for three months. The entire structure was found to be satisfactory, except that all the " Bulldog " connector bolts were slack and required tightening. This was due to the new timber drying out and consequently shrinking. The maximum measured deflection of the main truss was now found to be a $\frac{1}{2}$ inch, and this increase can be attributed to the slight opening up of joints due to the shrinking of the timber and the addition of the curtain loads. Thus a certain small further increase of deflection may be expected until the new timber becomes fully dried out.

The work was carried out for the Northampton Theatre Syndicate, the architects being Sir John Brown, A. E. Henson & Partners, and the Consulting Engineers, Messrs. Leonard & Grant. The main contractor was Messrs. T. Wilson & Sons.

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed.

To the Editor of THE STRUCTURAL ENGINEER.

The Fire Endurance of Timber Beams and Floors

Sir—Our attention has been drawn by Mr. C. F. Fischl, of the Joint Fire Research Organisation, to a simplified method of calculating the fire endurance of timber beams. In the paper referred to* a family of curves is used (Figs. 7 and 8) to obtain the fire endurance (t) for various sizes of beams with different values of the

$$\text{ratio } \frac{\text{applied load}}{\text{ultimate load}} \ (r).$$

By plotting from the experimental results given in Table 1 of the paper values of t/b against $3\sqrt{r/d^{\frac{1}{2}}}$ where b is the breadth and d the depth of the beam in inches, a straight line is obtained, the equation of which is

$$t = b\,(23.8 - 52.8\,(r/d^{\frac{1}{2}})^{\frac{1}{4}})$$

This gives the probable fire endurance in minutes.

A second straight line can be drawn on the graph to represent the minimum fire endurance of the beams tested and this time is given by the expression

$$t_{mm} = b(23.8 - 62.5(r/d^{\frac{1}{2}})^{\frac{1}{4}}).$$

Yours, etc.,

D. I. LAWSON, L. A. ASHTON, C. T. WEBSTER.

Boreham Wood,
Herts.
May 26th, 1953.

To the Editor of THE STRUCTURAL ENGINEER.

Vacuum Concrete

Sir—I wish to add my vote of thanks to Monsieur I. Leviant for his most interesting and informative paper† on the possibilities of extending the use of the vacuum process to concrete works.

There is, however, one point which is not clear and this is the number of connections required between the pump and the filters to obtain efficient results. As far as can be seen from the illustrations, the floor mats have a single hose connection direct to the pump but the number required for columns, circular or rectangular, cannot be judged. In dealing with the latter, is it necessary to connect the pump to a vacuum bottle and have multiple connections from it to the periphery of the column—and if so, how many ?

I shall be obliged if this point can be cleared up and I should also like to know details of the " special device " referred to in Fig. 13. A device which will give a perfectly smooth face to concrete without the cost of steel trowelling has vast possibilities.

Yours faithfully,

H. W. BARRAS (Member)

The Author replies :—

1. NUMBER OF CONNECTIONS BETWEEN PUMP AND FILTER.

It is usual to lead a pipe from the pump to a manifold, *i.e.*, a special piece of pipe with several outlets. From each of these outlets a smaller diameter pipe connection leads to the vacuum shutters or mats. The number of points on the mat or on the shutter through which the connection with the vacuum chamber (narrow space between the impervious shutter and the filter) is made varies with many factors such as the size of the shutter, thickness of the vacuum chamber, etc. Definite rules have, however, been devised by the specialist designers. In most cases where several of these points are required the connections are to a certain extent built in, so that only one outlet, and therefore only one pipe connection, is necessary on each element of shuttering.

2. SPECIAL DEVICE.

The Société du Vacuum Concrete has developed a new method of casting using a thin sheet called " Vivac." Instead of oiling the shutter, this sheet is placed on the inside as a lining and gives a polished appearance to the concrete. It serves to provide a surface that is much smoother than any trowelled concrete. After stripping, the Vivac sheet is peeled off and re-used many times.

To the Editor of THE STRUCTURAL ENGINEER.

Straub's History of Civil Engineering

Sir—With reference to Mr. P. M. Tezner's letter in THE STRUCTURAL ENGINEER for August, 1953, I would like, in justice to Dr. D. S. Stewart, to point out that he did *not*, in his book on the subject, forget to mention Professor Winkler as the inventor of Influence Lines ; the reference will be found at the beginning of Chapter 4.

Yours, etc.,

LESLIE GORDON (Associate Member).

Edinburgh.
August 13th, 1953.

Mr. TEZNER replies : I wish to convey my apologies to Dr. Stewart, pleading—not as an excuse—but perhaps as extenuating, that his reference to Professor E. Winkler appears to be ill-placed at the tail end of a sentence naming three other authors not connected with influence lines, alike the whole chapter referred to.

Owing to local conditions I had no exact knowledge of Dr. Hamilton's illuminating lecture on History of Structural Analysis, covering an important part of Hans Straub's book ; nor of the following discussion of the matter in the I.C.E. I should have referred to these in the letter to the Editor written in April last. I should have been particularly pleased to record Dr. Skempton's approval of Mr. Straub's endeavour, referring to the, then, only available Basel edition mentioned in an annotation to the discussion.

*THE STRUCTURAL ENGINEER. Vol. XXX, No. 2 pp.27-33, Feb. 1952.

†THE STRUCTURAL ENGINEER. Vol. XXX, No. 11, pp. 249-258, Nov. 1952.

Institution Notices and Proceedings

FORTHCOMING MEETINGS

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, November 12th, 1953

Joint Meeting with the British Section of the Société des Ingenieurs Civils de France, at 6 p.m., when Monsieur L. P. Brice will give a paper on "Different Types of Wharves and Maritime Structures recently executed in France."

A limited number of advance copies of the paper will be available and may be obtained on application from the Hon. Secretary of the British Section of the Société des Ingenieurs Civils de France, Mr. W. R. Howard, M.I.Struct.E., 82, Victoria Street, London, S.W.1.

Thursday, November 26th, 1953

Ordinary General Meeting for the election of members at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E. (Member of Council), and Mr. K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E., will give a paper on "The Reconstruction of a Soaking Pit Building."

Thursday, December 17th, 1953

Ordinary General Meeting, at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. W. Hamilton, A.M.I.C.E., and Mr. G. P. Manning, M.Eng., M.I.C.E., will give a paper on "Some Structural Uses of Aluminium Alloy with special reference to Domes."

Thursday, January 14th, 1954

Ordinary General Meeting at 6 p.m., when Mr. S. J. Crispin, L.R.I.B.A., M.I.Struct.E., will give a paper on "Soil Stabilisation of Fine Materials."

Thursday, January 28th, 1954

Ordinary General Meeting at 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. E. J. Callard, M.A., M.I.Mech.E., will give a paper on "Design and Construction of a Welded Portal Frame Warehouse Building, Designed by the Plastic Method."

Members wishing to bring guests to the Ordinary and Joint Meetings referred to above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS—JANUARY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 5th and 6th (Graduateship), and January 7th and 8th, 1954 (Associate-Membership).

EXAMINATION RESULTS. JULY, 1953

Home Centres

The Examinations were held in July, 1953, at the usual centres in Great Britain. One hundred and five candidates took the Graduateship Examination, and 309 took the Associate-Membership Examination, making a total of 414. Of these, 55 passed the Graduateship Examination and 79 passed the Associate-Membership Examination.

The names of the successful candiates are :—

Graduateship Examination

Barneveld, Tom Harris, Bartley, Raymond, Biss, Russell George, Bradley, Dennis, Brett, Peter Ronald, Bromige, Keith Bernard, Burnell, Ian Derek, Burton, Thomas Edmund, Carnan, Lewis Rippon, Chorley, Victor Wilfred, Clark, John Alan, Connelly, Terence Joseph, Corbett, Joseph Frank, Coverman, Sidney Harold, Cross, John Henry, Czoske, Jan Edmund, Dorey, Conrad Webster, Dunn, Horace William James, Egwuatu, Gordon Chukwuka, Ellerington, Charles Richard, Feakes, Gerald William Ernest, Freestone, Geoffrey, Hawkes, John Murray, Hookham, Raymond Albert, Hunter, Kenneth, Idowu, Charles Olatunji, Jablonski, Piotr, Kalnins, Janis Osualds, Kay, Terence, Kirby, Norman, Morgan-Green, Peter, Myers, Leslie Walter, Nash, Alan Blake, Newby, Ronald William, Parish, Ancel, Pinkney, John Dennis, Pivirotto, Witold, Pringle, Alan, Reffitt, Michael John Joseph, Sharples, Kenneth, Silber, Ronald Walter, Smart, Frederick Arthur, Smith, Raymond Charles Henry, Taratula, Ryszard Leopold, Thomas, David, Todd, Paul, Underwood, Bernard George, Vickers, Rex Adrian, Wagner, Arthur Hiram, Wellings, Paul Eustace, Wilks, Leslie Robert, Winfield, Peter Frederick, Wing, Arthur James, Wingate, Gerald, Woollard, Malcolm William Thomas.

Associate-Membership Examination

Adasko, Stanislaw, Anderson, Jack Ernest, Bailey, John, Beck, Roy Cecil, Binder, Paul Frank, Bond, Derek, Boswell, Peter, Bowering, Brian Arthur, Brown, Cecil Allen, Brown, David Henry, Bullock, Leslie, Cobb, Walter Clifford, Conway, Gerald Ernest, Coppen, Frank, Cording, John Stanley, Creighton, Leslie, Dickinson, John George, Drummond, Richard, Evans, Maurice Hambling, Farmer, Lawrence Theodore, Fasehun, Ebenezer Olawanle Oladipo, Gaunt, Grenville, Gawler, Stanley William, Hall, Thomas Robert, Hansen, John, Haque, Mohammed Manzurul, Hinder, Stanley James, Holmes, Ian Miles, Holt, Hermann Paul, Huggins, Raymond Arthur, Jackson, James Keith, Joshi, Aravinda Vithal, Kirkman, Cyril John, Krarup, Anthony Charles, Leach, Alan Brickett, Lewis, Frank Edgar, Loadsman, Roy Vernon Charles, Longman, Donald James, Lord, Peter Bamford, Macintyre, John Ross McDermit, Masters, Patrick Anthony, Mayo, George Edward, Milligan, William Ramsay, Morrell, Donald Beaumont, Morris, David Austin, Murphy, William, Overton, Kenneth Albert, Parsons, Dudley John Francis, Power, Leslie Brian, Preece, Stanley George, Puller, Malcolm John, Pun Yin Keung, Purvis, John Ferguson, Rae, Richard George, Raggett, Leonard John, Redlich, Walter, Riley, Thomas William, Robinson, Ernest Arthur Feakes, Ross, Colin Forrester, Sampson, Derick Stanley Maxwell, Sawyer, Robert Arthur, Sayers, Ronald Ernest Courtney, Scholnick, Gabriel, Shippen, Clive, Silhan, Stanley George, Simpson, Morrison, Slee, John Penry, Smart, James Desmond, Smedley, Granville Barrie, Somerset, Francis Eyre, Spoors, Colin, Tate, Alan George, Walden, Kenneth Owen, Walker, Charles Henry, Walmsley, John, Wattie, Neville, Webster, Stanley John, Willcox, Leslie George, Wright, Frank.

MACLACHLAN LECTURE COMPETITION

The closing date for the receipt of entries for the MacLachlan Lecture Competition is Wednesday, March 31st, 1954. Particulars of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture, and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures.

3. Entrance into the competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

PARTICULARS OF THE COMPETITION FOR 1954

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1954.

2. The subject of the Lecture shall be on any aspect of structural engineering.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition, will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Wednesday, March 31st, 1954.

LONDON GRADUATES' AND STUDENTS' SECTION

The next meeting of the Section will be held at 11, Upper Belgrave Street, S.W.1, on Tuesday, November 10th at 6 p.m., when Mr. J. F. S. Pryke, B.A. will give a lecture on "Underpinning." In his lecture, Mr. Pryke will discuss underpinning generally, foundations on London clay, investigating London clay to find a suitable bottom, the apparatus used for testing soils and a new method of underpinning enabling long lengths of wall to be pinned in one operation.

A meeting will be held on Tuesday, December 8th, when Mr. S. Turner will give a paper on "The Foundation of an 8,000,000 cu. ft. Gasholder."

Lectures and "lecturettes" by members of the Section are always welcome and any member wishing to read a paper should communicate with the Hon. Secretary, J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

INTERNATIONAL ASSOCIATION FOR BRIDGE AND STRUCTURAL ENGINEERING

The Executive and Permanent Committees of the International Association for Bridge and Structural Engineering held meetings at Zurich on June 22nd-25th. The President of the Association, Professor F. Stussi, of Zurich, was in the chair, and was supported by Mr. Ewart S. Andrews, the British Vice-President, and Monsieur L. Cambournac, the French Vice-President. Sixty-seven members of the Committees were present, representing fourteen countries, the British members, in addition to Mr. Andrews, being Mr. C. S. Chettoe, Mr. O. A. Kerensky, Mr. H. Shirley Smith, Mr. P. G. Bowie, and Mr. F. S. Snow.

Dr. Moritz Klonne, of Dortmund, was re-elected Vice-President representing Germany, a position he held before the war.

An invitation was received from Professor E. Torroja to hold the next Committee meetings in Madrid in September, 1954. Professor J. Balard da Fonesca, who had invited the Association to hold the next Congress in Lisbon in 1956, was elected Vice-President.

The question of finance and the method of presentation of the publications were among the other matters discussed.

On June 24th and 25th, the members visited the engineering works over the Sustenpass and the hydro-electric power stations at Oberhasli.

BRANCH NOTICES

LANCASHIRE AND CHESHIRE BRANCH

The following meetings have been arranged :—

Thursday, November 26th, 1953

At the College of Technology, Manchester, 6.30 p.m. Three short papers by younger members of the Branch.

Friday, December 18th, 1953

Annual Dinner and Dance, at the Grand Hotel, Manchester.

Tuesday, January 12th, 1954

Joint Meeting with the Institute of Welding, Liverpool and District Branch, at the College of Technology, Liverpool, at 7.15 p.m. A paper on "The Plastic Theory and its Application to Welded Structures," will be given by a member of the staff of the British Constructional Steelwork Association.

Wednesday, January 27th, 1954

Joint Meeting with the Institution of Civil Engineers, North-Western Association, at the College of Technology, Liverpool, at 6.30 p.m., when Mr. Edgar Morton, M.Sc., will give a paper on "Site Exploration and Foundation Problems."

Joint Hon. Secretaries : A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire ; M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, November 10th, 1953

At the Supper Room, The King's Hall (Queen Street Baths), Queen Street, Derby, 7 p.m., when Mr. E. McMinn will give a paper on "Tubular Structures."

Friday, November 27th, 1953

Joint Meeting with the Birmingham and Five Counties Architectural Association, when Mr. F. J. Samuely, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., will give a paper on "Design and Application of Space Frames and Skin Structures," at the James Watt Memorial Institute, Birmingham.

Friday, January 22nd, 1954

At the James Watt Memorial Institute, Birmingham, at 6.0 p.m. Mr. W. G. Carter, M.B.E., B.Sc., A.M.I.C.E., A.M.I.Struct.E., will give a paper on "True Economics in Planning, Designing and Detailing of Structures."

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Monday, November 30th, 1953

At the James Watt Memorial Institute, Birmingham, 7.0 p.m. Mr. V. H. Lawton, M.I.Struct.E., on "Plastic Design."

Friday, January 29th, 1954

Details to be announced.

All meetings to be held at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7 p.m., unless otherwise stated.

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, November 3rd, 1953

At Middlesbrough, Mr. L. R. Creasy, B.Sc., M.I.C.E., on "The Use of Structural Steel in Buildings."

Wednesday, November 4th, 1953

The above meeting will be repeated at Newcastle.

Tuesday, December 1st, 1953

At Middlesbrough, Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E. (Member of Council), and Mr. K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E., on "Reconstruction of a Soaking Pit Building."

Wednesday, December 2nd, 1953

The above meeting will be repeated at Newcastle.

Wednesday, January 6th, 1954

At Newcastle. Joint Meeting with The Northern Architectural Association.

Wednesday, January 13th, 1954

At Middlesbrough. Joint Meeting with the Institution of Civil Engineers. Mr. A. P. Clark on "Lackenby Works."

All meetings commence at 6.30 p.m., the Middlesbrough meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and those at Newcastle in the Neville Hall, near the Central Station.

Hon. Secretary : Captain O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, November 3rd, 1953

Joint Meeting with the Northern Ireland Association of the Institution of Civil Engineers. Mr. Donovan H. Lee, B.Sc., M.I.C.E., M.I.Mech.E., M.Am.Soc.C.E. (Member of Council), on "The Advantages and Disadvantages of Structural Steelwork, Reinforced Concrete and Prestressed Concrete." At 6.30 p.m.

Tuesday, December 1st, 1953

Paper by Mr. R. J. N. Sweetnam, M.A., M.A.I., A.M.I.C.E. (Graduate).

Tuesday, January 12th, 1954

Details to be announced.

Unless otherwise stated, meetings will be held at the College of Technology, Belfast, at 6.45 p.m., preceded by tea at the Overseas League premises, Wellington Place, Belfast, at 6 p.m.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Tuesday, November 17th, 1953

At Ca'doro Restaurant, Glasgow, at 6.0 p.m. Mr. V. H. Lawton, M.I.Struct.E., "Examples of Structural Steel Design to B.S.449."

Thursday, December 3rd, 1953

At Elite Hotel, Windsor Street, Edinburgh, at 6.0 p.m. Mr. James E. Richey, M.C., B.A., Sc.D., "Alkali Reaction in Concrete."

Tuesday, December 15th, 1953

At Ca'doro Restaurant, Glasgow, at 6.0 p.m. Details to be announced.

Monday, January 18th, 1954

At Ca'doro Restaurant, Glasgow, at 6.30 p.m. Details to be announced. Visit of the President and the Secretary of the Institution.

Tuesday, January 19th, 1954

Annual Dinner and Dance at Grosvenor Restaurant, Gordon Street, Glasgow. The President and the Secretary of the Institution will be present.

Hon. Secretary : G. Drysdale, A.M.I.Struct.E., "Niaroo," 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Saturday, November 7th, 1953

At the Duke of Cornwall Hotel, Plymouth, at 6.30 p.m. Chairman's Address, by Colonel R. Hazzledine, O.B.E., T.D., M.I.Struct.E. The President and the Secretary of the Institution will attend the meeting, which will be followed by a Dinner to mark the Coronation Year.

Friday, December 11th, 1953

At Exeter, at 7.0 p.m. Mr. H. J. Scoles, L.R.I.B.A., M.I.Struct.E., on "Gap Graded Concrete."

Friday, January 15th, 1954

At Newton Abbot, at 7.0 p.m. Mr. C. J. Woodrow (Graduate) on "The Tamar Bridge Proposal."

Hon. Secretary : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, November 4th, 1953

At Swansea. The Chairman, Dr. A. A. Fordham, M.I.Struct.E., A.M.I.C.E., will give an Address entitled "Research Work in Soil Mechanics at the University College of Swansea." The Address was given at Cardiff on October 28th. On this occasion the Address will be followed by a discussion.

Saturday, November 7th, 1953

At Colwyn Bay. The Chairman's Address will be repeated and will be followed by discussion.

Monday, November 30th, 1953

At Cardiff. Joint Meeting with the Institute of Welding, Mr. S. M. Reisser, B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E., on "Recent Developments in Welded Structures."

Tuesday, December 8th, 1953

At Swansea. Meeting followed by films.

Tuesday, January 26th, 1954

At Cardiff. Structural Quiz.

Wednesday, January 27th, 1954

At Swansea. Structural Quiz.

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel, at 6.30 p.m.

Meetings at Colwyn Bay will be held at the County Buildings, at 6 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.CE., A.M.I.Struct.E., 2, Celtic Road, Gabalfa, Cardiff.

WESTERN COUNTIES BRANCH

Friday, November 6th, 1953

Combined meeting with the Institution of Civil Engineers. Professor A. G. Pugsley, O.B.E., D.Sc., F.R.S., M.I.C.E., F.R.Ae.S. (Vice-President), on "Suspension Bridge Development illustrated by some Bristol Avon Bridges."

Friday, November 27th, 1953

Mr. L. Scott White, O.B.E., M.I.C.E. (Past President), on "Moving King Henry VIII's Wine Cellar, Whitehall."

Friday, December 4th, 1953

At the Royal Hotel, Bristol. Combined Dance.

Friday, January 8th, 1954

Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., on "Hangars at London Airport—Design of Large Span Prestressed Concrete Beams."

Unless otherwise stated, all meetings will be held in the University of Bristol Geology Lecture Theatre, at 6.0 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, November 18th, 1953

Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., on "Prestressed Concrete Beams at London Airport."

Wednesday, December 16th, 1953

Mr. W. N. Espie, A.M.I.Struct.E., on "The Handling of Bulk Cement."

Wednesday, January 20th, 1954

Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E. (Member of Council), and Mr. K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E., on "The Reconstruction of a Soaking Pit Building."

The above meetings will be held at the Great Northern Hotel, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone; 34-1111, Ext. 257.

Natal Section, Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary : R. Stubbs, M.I.Struct.E. The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

Book Review

Theory of Perfectly Plastic Solids, by W. Prager and P. G. Hodge, Junr. (New York : Wiley ; London : Chapman & Hall, 1951.) 6 in. × 9¼ in., pp. 264. Price $5.50.

This essentially theoretical work, which except for illustrative purposes does not attempt to apply the theory to practical problems, will be primarily of interest to research workers. It provides an introduction to the broad field of plastic theory underlying modern conceptions of structural failure whether in the realm of soil mechanics or frames, and to quote the authors, is not an exhaustive treatise on the theory of plasticity but rather an introduction at intermediate level to the theory of perfectly plastic solids. It neglects such phenomenon as strain hardening, but due to this severe limitation in scope the authors are able in a concise text-book to cover briefly the fundamental conceptions relevant to a true understanding of the recent work on structural failure.

After introducing the basic concepts of the plastic theory the elastic plastic behaviour of a simple truss and beam are now examined from the theoretical aspect, and this is followed by a chapter on torsion which forms an introduction to the general theory of plane strain. Examples of incipient and steady plastic flow, which indicate possible applications of the theory, are then outlined, and after considering limit analysis, the use of extremum principles is discussed. The inclusion of problems in an excellent advanced text-book of this nature, which does not shrink from the use of advanced mathematics where necessary, does however appear unnecessary.

A. R. G.

Some Structural Uses of Aluminium Alloy with Special Reference to Domes*

By W. Hamilton, A.M.I.C.E. and G. P. Manning, M.Eng., M.I.C.E.

Synopsis

A record of the design, development and construction of the first long-span aluminium domes by the two engineers who were solely responsible for them. Their experiences in dealing with the first dome, which was the lightest structure in the world and with the first oil tank roof which was the largest all-welded aluminium structure in the world.

Introduction

This is in no sense a collection of information but an account of personal experiences and personal reactions. General conditions are only discussed in so far as they clarify or explain the main subject-matter.

Choice of Alloy. Aluminium may be alloyed with small percentages of copper, iron, magnesium, manganese, nickel, silicon or zinc. There have been several different nomenclatures for each standard alloy and some alloys may be artificially strengthened by work-hardening, others by heat treatment. Add to this that

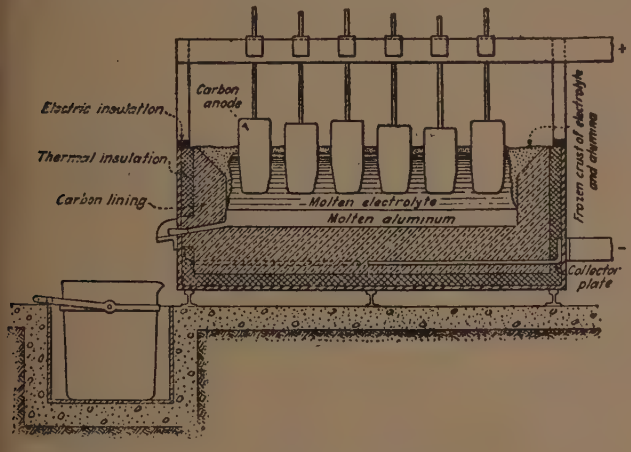


Fig. 1.—Reduction furnace

sections may be produced by extrusion, by rolling or by drawing, and the engineer wishing to gain his first experience of aluminium as a structural medium is faced with a bewildering array of letters, figures and processes. To avoid this, only one alloy will be described here at the outset. The most popular alloy at the moment for structural sections is one containing silicon (about 1 per cent.) and magnesium (about ¾ per cent.) double heat-treated, and most of the structural sections described in this paper were of this alloy. It was described by the makers as 51ST but was more generally known as AW10B, and is now officially classed in B.S.1476 : 1949 as HE10WP.

It has the following mechanical properties.

0.1% Proof Stress	typical	18 tons per sq. in.
	guaranteed	15 ,, ,, ,,
Ultimate Tensile	typical	20 tons per sq. in.
	guaranteed	18 ,, ,, ,,
Elongation on 2-in.	typical	13%
	guaranteed	10%
Elastic Modulus ...	...	9,500,000 lb. per sq. in.
Ultimate Shear Stress ...	...	14 tons per sq. in.
Bearing Yield Stress ...	...	20 tons per sq. in.
Ultimate Bearing Stress	...	36 tons per sq. in.
Compressive Proof Stress	...	18 tons per sq. in.

A description of the method of measuring the first three of these values is given in B.S.1476. The last four items are described in Appendix A.

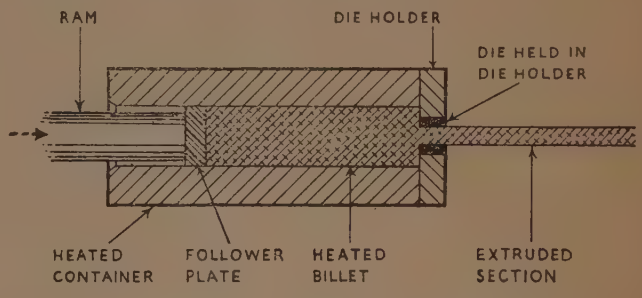


Fig. 2.—Principle of operation of the extrusion press

B.S. 1476: 1949. This is one of a series numbered 1470 to 1477 and 1490. Unfortunately this B.S. repeats the fundamental fault of B.S.15 (Mild Steel) in failing to specify any minimum properties in compression. Regrettable as this lapse may be in B.S.15 it is still more deplorable in B.S.1476, since the relatively small elastic

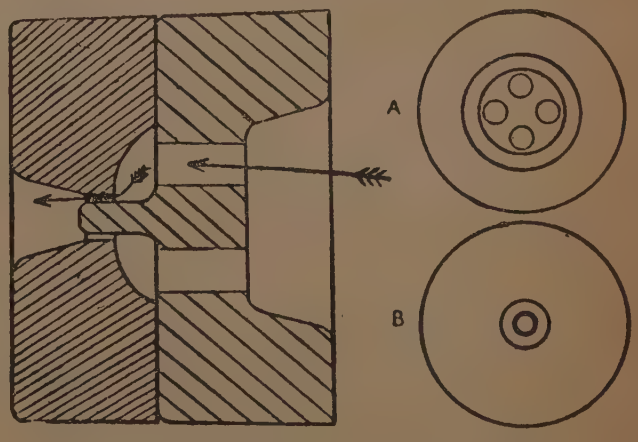


Fig. 3.—Diagram of ante-chamber die
A. Back view. B. front view.

modulus of aluminium militates strongly against its use for compression members. Since this paper deals mostly with shallow domes, which are basically in compression, the vital nature of this omission will be apparent. Whether the issuing of the present spate of

To be read before the Institution of Structural Engineers at 11, Upper Belgrave Street, London, S.W.1, on Thursday, December 17th, 1953, at 6 p.m.

British Standards is a blessing or a curse to the industry is a point open to argument.

Production of Structural Sections in HE10WP

Despite the vast amounts of aluminium in clays and allied rocks, extraction is only commercially possible from bauxite. Alumina (aluminium oxide) is prepared from bauxite and is reduced in electric furnaces (Fig. 1) to give a metal which is about 99.0 per cent. pure aluminium. This is alloyed with silicon and magnesium to give the specified composition and cast into cylindrical billets to suit the diameter of the extrusion press (Fig. 2). Presses up to 15-in. diameter are in use in this country and any section which will fit inside such a circle (e.g., a 12 in. × 5 in. joist) may be produced. Hollow sections may be made by machining a hole through the billet and passing a shaped steel mandrel through this hole. The section shown in Fig. 13 was actually produced by this process. Apart from the cost, when a complicated section is required this method has certain technical difficulties and hollow sections are now made alternatively by using a bridge die as in Fig. 3. The bloom is heated to about 500° C. before extrusion commences. This, and the heat generated by forcing the plastic metal through the holes, causes it to reunite after passing through the holes in the primary part of the die, thus giving a hollow section with continuous walls. Unfortunately, this re-uniting is not always complete and a small percentage of early hollow extrusions showed lines of weakness. This would be a matter of great concern in the design of small Hamman type domes, since small hollow sections are used. Fortunately, the considerable strains produced by curving the ribs pick out any weak specimens. Out of the material for half-a-dozen domes some three or four lengths of tube, on being passed through the curving rolls, disintegrated into four separate angles.

After leaving the press the sections may not be straight. The smaller ones may be stretched into line and the larger ones passed through straightening rolls. The metal is still in the soft state with a tensile strength of about 8 to 10 tons per sq. in. It is next heated to

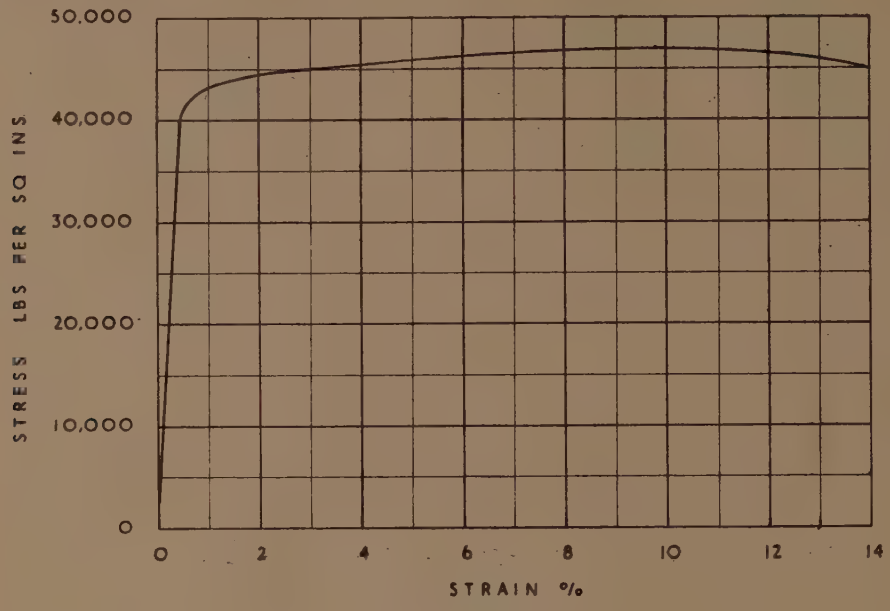


Fig. 4.—Tension

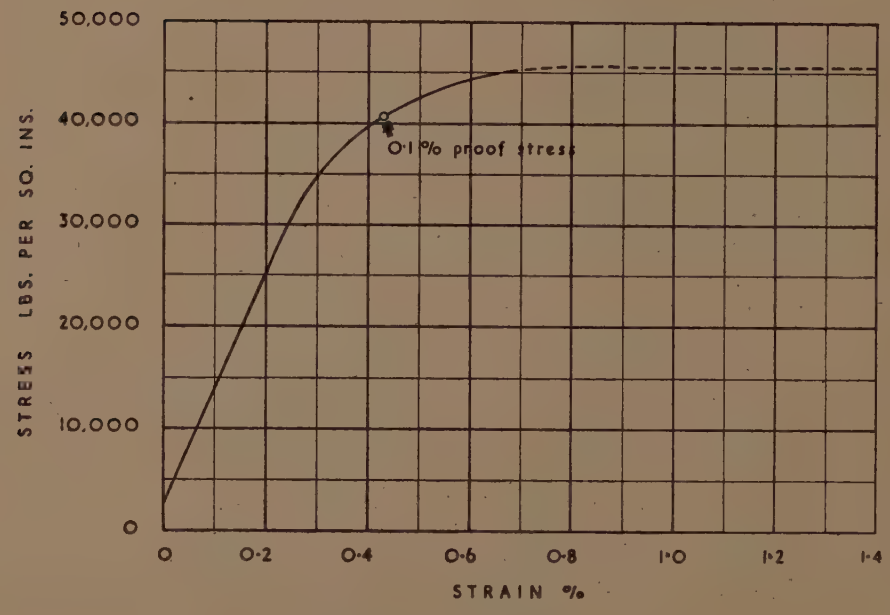


Fig. 5.—Compression

320°C. for about 45 minutes and then quenched in cold water. It is still soft but after standing at normal room temperature for some days its strength reaches 12-15 tons per sq. in. It is now again heated to 175°C. for about five hours and allowed to cool, this second

$$\text{The L.U.D. } \Delta = \frac{L^2}{1{,}090a}$$

where L = equivalent round-ended length.
a = overall depth of section.

Fig. 6.—Aluminium shuttering, footbridge, etc.

heat treatment finally increasing the tensile strength to 18-20 tons per sq. in. The letters and figures HE10WP describe these processes as follows :—

10 means an aluminium-silicon-magnesium alloy within the limits of B.S.1476.

H means that this alloy is amenable to heat treatment.

E means that it is in the form of an extrusion.

W that it has undergone the first (Solution) heat treatment.

P that it has also undergone the second (Precipitation) heat treatment.

Mechanical Properties of HE10WP

Figs. 4 and 5 show typical tensile and compression tests. From these a practical limit of proportionality

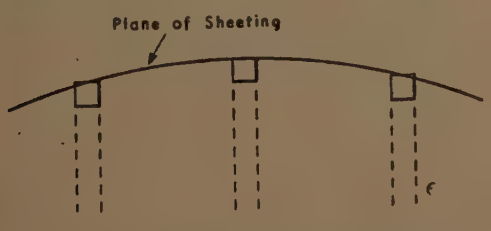


Fig. 7.—Ribs lying in vertical planes

of ±35,000 lb. psi with E = 9,500,000 lb. psi may be assumed.

A safe stress of 17,500 lb. psi is indicated for this specimen but as a downward variation of 10 per cent. is allowed (see B.S.1476) this would become 15,700 lb. psi or 7 tons psi.

$$\text{L.U.D. factor} = \left(1 + \frac{1}{2{,}180}\left(\frac{L}{k}\right)^2\right)$$

Standard Structural Sections to B.S.1161

This British Standard has been attacked on the ground that an engineer, having for years been limited

Fig. 8.—Tenon joint in square tube

to the standard rolled sections for structural steelwork, may subconsciously assume that his choice of aluminium structural sections is limited to those shown in this B.S. The cost of a new die for any non-hollow section is quite small. Not only is it easily possible to have angles, channels and joists with non-standard thicknesses and

non-standard widths, but entirely non-standard shapes such as the " top-hat " may just as easily be extruded. With an expensive metal such as aluminium, the use of any over-size section is most uneconomical and the design of individual sections to meet every individual case (as in reinforced concrete) is urged. From this point of view the criticism is legitimate, but firstly the tabulated values for I and Z are a great help in choosing a preliminary standard section which may then be modified to suit a particular case. Secondly, it is

Fig. 9.—Butt joint between tubes

unreasonable to avoid using a section merely because this resembles a standard steel section. Lastly, if a small amount of section is wanted quickly it is much cheaper to take an over-strong standard section than to incur the expense and delay of obtaining a special die.

Workshop Characteristics of Structural Alloys

Apart from an occasional sharp corner on the sheared edges of plate, aluminium alloys are easy to handle and machine. The surface will take a pencil line and centre-punches clearly. The metal drills and machines best at higher speeds than those suitable for mild steel. It is easy to handle and quite sizeable jobs (e.g., 40 ft.

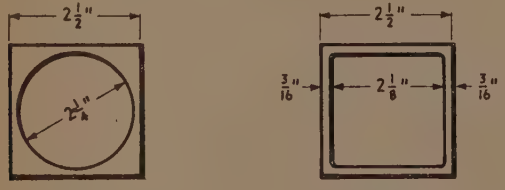


Fig. 10.—Hollow tube sections

Fink trusses) can be put together in a small shop without overhead mechanical lifting appliances and without radial drills.

Tolerances in thickness and shape are greater than in steel sections. Angle sections tend to be less than 90°. These variations are not sufficient to create any difficulties in normal structural work, although they are important in dome fabrication. It is possible to have them reduced by re-rolling or re-drawing but the process is expensive and the authors have not seriously considered it.

Welding Aluminium Alloys

Some of the structures described were welded by Argon-arc and some by the Murex process. One of the difficulties of welding aluminium is its very high thermal conductivity. This makes it very difficult to weld a very thin member to a thick one, for the thicker member conducts away the heat so quickly that the thinner one melts before the thicker one reaches welding heat. This may be overcome by heating the thicker members by bunsen jets. But the most serious drawback to welding heat-treated alloys is the reduction in strength caused by the welding heat. The strength of HE10WP when fully annealed drops to one-third and all important welded work must be checked by testing specimen welds.

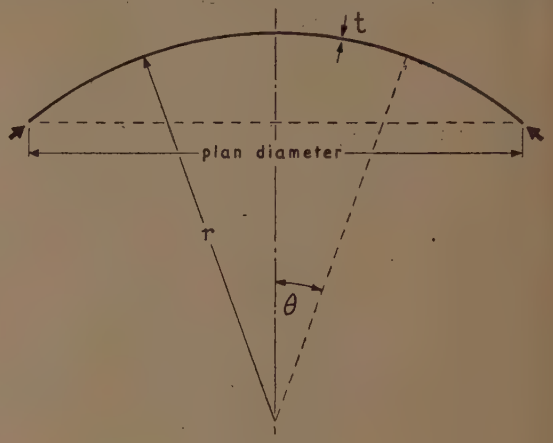


Fig. 11

Gas welding was successful in joining 8 in. diameter pipes but for light work it was found that distortion became a problem.

Bolted and Riveted Joints

There are several systems of riveting which are satisfactory for fastening sheeting to main members.

The simplest way of assembling light structures such as roof trusses is by sherardised steel bolts.

It is surprising that assemblage of light and medium structures by aluminium rivets has not been more widely developed. Rivets of 16S alloy ½ in. diameter in double shear in ¼ in. HE10WP flats fail at 5 tons each. These

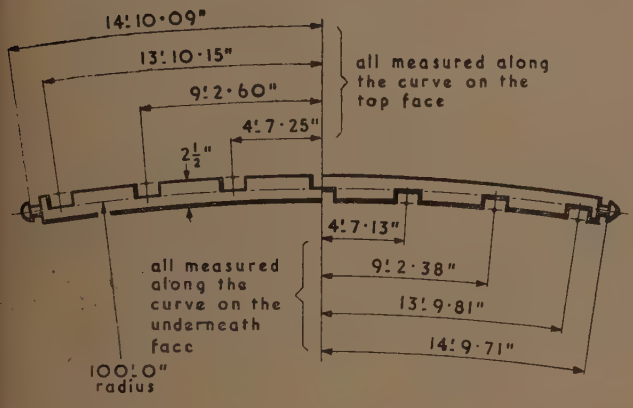
Fig. 12.—10 ft. × 10 ft. 16-gauge sheet loaded with 16-in. sand

may be closed by pneumatic hammer or by a 20-25 ton squeeze. Rivets ¾ in. diameter failing at 11 tons each could be closed by a 40-ton squeeze. They must, of course, be closed cold. Drilling for these rivets must be clean and accurate and only slightly over size (say +1/32 in.). To give a neat head the length of rivet must be carefully chosen.

Fancy types of rivet head that need less closing have been developed but this seems rather begging a question

which could be easily dealt with by a little initiative on the part of any well-equipped structural shop. Some part of each sherardised bolt or washer is unavoidably stripped of its zinc coating when being screwed up tight and rusting commences.

Fig. 13.—Test failure of tenon joint



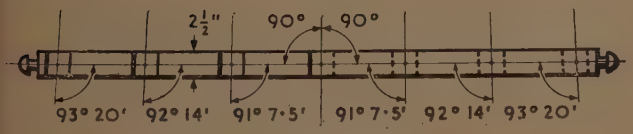


Fig. 14.—Above—Half elevations
Below—Half developed plans

Aluminium alloys do not make smooth running threads like mild steel and aluminium bolts are a bit "sticky." Bolts in the softer alloys may be used to assemble very light work but bolts in the very strong alloys are expensive and seem very difficult to obtain.

Long Span Aluminium Domes—History

Most new forms of construction are arrived at by successive small extrapolations from existing and established forms. During this process many engineers take part and share between them the ultimate credit. Long-span aluminium domes provide a complete contrast to this general state of affairs. The Hamman method of construction is not in any way based on any

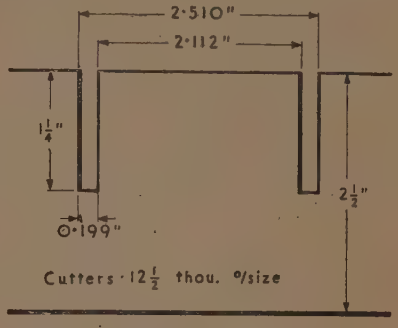


Fig. 15.—Standard tenon cut

preceding method. The final details of the first long-span dome were worked out at the outset and adhered to throughout. Only two engineers had any hand whatever in the design, development and construction of the first dome which was so essentially correct that identical details were followed for six further domes. As far as the authors can ascertain no form of construction resembling an aluminium dome had been attempted or even contemplated before the first Hamman dome was completed and any credit there may be for this form of construction must therefore belong to them alone.

Mr. Hamilton in 1946 developed and patented a method of shuttering for reinforced concrete and used HE10WP for the structural parts thereof (Fig. 6). This led to various other small uses (Fig. 6). In 1945 Mr. Frank Tipler, M.I.M.E., suggested to Mr. Hamilton that

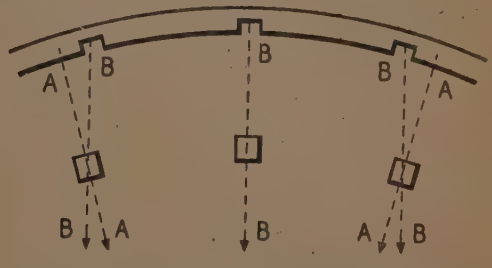


Fig. 16.—Joints mating
Lines A-A show correct direction of approach
Lines B-B show actual direction of approach

the roofs of sour crude oil tanks could be built in aluminium alloy. On November 30th, 1945, the authors met, and Mr. Hamilton suggested that such roofs could be built as shallow domes. The sketch which he then made is still in existence.

Some engineers have hazarded a guess that the Hamman arrangement of members was worked out on some complicated theory of stress distribution. Others have guessed that it was based on some preceding type. Both guesses are entirely wrong. The reason for its adoption was much simpler and much more elementary. It was clear at the outset that the cost of the metal would be a vital factor and that thin sheeting was

essential if the cost were to be kept within reasonable limits. As these domes would need to be vapour-tight some simple arrangement of sheeting was necessary or the difficulties of making a vapour-tight joint at the corners where individual sheets overlapped might prove insuperable. It was clear that the sheeting would be strongest and simplest if all the panels could be made square, and it was this idea alone that led to the Hamman

Fig. 17.—Complete 82 ft. 6 in. dome packed on lorry

framing. Curiously enough, when the first vapour-tight dome came to be built the difficulties of overlapping sheets was circumvented and did not arise.

Principles of Hamman Construction

The original idea had been to make a grid of members square on plan, each rib consisting of a circle lying in a vertical plane. This would have meant that the radius of curvature of the ribs would vary. When the first idea of using circular tubes was abandoned a further difficulty arose, for this arrangement made it impossible

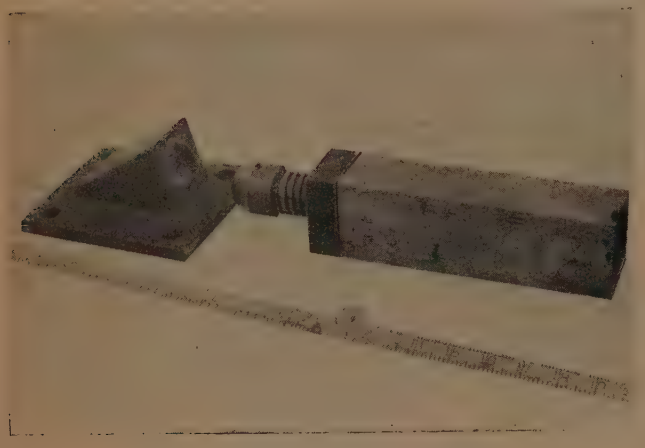


Fig. 18.—End adjusting screw

to use square section tubes since their upper surfaces would lie always in a horizontal plane and not in the plane of the sheeting (see Fig. 7). Both these difficulties were overcome by locating all the ribs on two series of great circles.

If we draw the whole sphere of which our dome is a part and refer to the crown of the dome as the north pole, then one series of ribs passes through the points (lat. 0°, long. 0°), (lat. 0°, long. 180°), the other series passing through the points (lat. 0°, long. 90°), (lat. 0°, long. 270°). As our dome is only a small part of the complete sphere the amount by which the ribs approach one another is small and the panels of sheeting are all approximately square and of approximately the same size.

Since these domes may be used to roof steel tanks and since the exact shape of the top of such tanks may vary from a true circle, it was decided to supply screw adjusters at both ends of all ribs (see Fig. 18).

The normal point at which joints are made in structural members is at the point where they meet one another. But in a spherical surface the design of such a joint is a three-dimensional problem and its exact

Fig. 19.—Erection of first ribs

fitting raises doubts and difficulties. It was therefore decided to make the ribs as far as possible continuous from springing to springing. Actually, half each rib runs through every joint, the ribs passing one another by double tenon cuts milled out of each (see Fig. 8). It is, of course, not practicable to fabricate or transport ribs over 80 ft. long and they are cut into lengths of about 20 ft., but the joints between these 20 ft. lengths are kept clear of the points of intersection and consist of plain square butt joints strengthened by inserting a smaller tube inside the main one (see Fig. 9).

Fig. 20.—Detail at thrust ring

To recapitulate, the main points of the Hamman dome are :—

1. Simple square panels of sheeting all one size and interchangeable.
2. All ribs on great circles.
3. Adjusting screws at all bearing shoes.
4. Ribs continuous through intersection points.
5. None of the major loads are carried through bolted, riveted or welded joints.

The construction is patented.

Dimensions of Prototype Roof

The first dimensions suggested for a prototype dome were 80 ft. plan diameter and 6 ft. rise. This latter was increased to 7 ft., giving a rib radius of about 117 ft.

Later it was decided to fix the radius of curvature of the ribs at exactly 100 ft., which further increased the rise to about 8 ft. 4 in. It was finally decided to increase the plan diameter to 82 ft. 6 in., which made the final rise about 8 ft. 11 in., or nearly one-ninth of the span. It was hoped to obtain sheets 6 ft. wide but only 5 ft. sheets were forthcoming and the basic spacing of the ribs was therefore fixed at 5 ft. centres.

It was first suggested that the ribs should be 3 in. diameter tubes but difficulties of jointing the tubes to one another and difficulties of attaching the sheeting

The unit compressive stress on a thin spherical dome of thickness t and uniform weight w is :—

(a) Towards the pole $\dfrac{wr}{(1+\cos\theta)\,t}$

(b) Round the lines of latitude $-\dfrac{wr}{t}\left(\dfrac{1-\cos\theta-\cos^2\theta}{1+\cos\theta}\right)$

where r = radius of sphere.
θ = angle measured down from the pole.

Fig. 21.—Erection of main ribs complete

ruled these out and by March, 1946, a $2\frac{1}{2}$ in. × $2\frac{1}{2}$ in. square tube had been decided on. At first the makers supplied one with a circular hollow as Fig. 10 but were later able to supply one with a square hollow, and this shape was finally used.

It was decided to use 16 gauge sheeting and although subsequent tests showed that 18 gauge was strong enough for all normal roof work the thicker sheets are easier to rivet or weld and sag less in the centres of the panels. (See Fig. 12.)

Design and Development of the Prototype Dome

In the design, fabrication and erection of lattice domes a number of problems arise. Some of these may be solved :—

(1) By theory.
(2) By experiments on models.
(3) By tests on full-scale members.
(4) Only by tests on a full-scale complete roof.

Method (2) was ruled out as small sections of HE10WP were unobtainable and the construction of a small model of such a light structure (the main ribs of a model 40 in. diameter would be only 0.1 in. × 0.1 in.) entailed such difficulties that the result could not be said accurately to resemble the full-scale roof.

Methods (1), (3) and (4) are described in the following paragraphs.

Elementary Theory of Uniform Dome Symmetrically Loaded

Any surface of revolution about a vertical axis supported on a horizontal thrust ring may, if symmetrically loaded by distributed loads, be regarded as supporting itself by purely direct tension or compression. (Fig. 11.)

The stress (a) increases as θ increases and is greatest round the edge of the roof.
The stress (b) decreases as θ increases and becomes zero when $\theta = 51°—48'$.

In the first dome actually erected at Sheerwater
r = 100 ft.
plan diameter = 82′ 6″.

Fig. 22.—Sheeting prototype dome

maximum value of stress (a) = $\dfrac{wr}{1.911t}$

maximum value of $\theta = 24°—22'$.

maximum value of stress (b) (at pole) = $\dfrac{wr}{2t}$

If w = 30 lb. per sq. ft. and if all the compressive thrust is taken by ribs spaced at 5 ft. centres

$$\text{maximum thrust per rib} = \frac{30 \times 100}{1.911} \times 5 = 7{,}850 \text{ lb. } (3\tfrac{1}{2} \text{ tons})$$

$$\text{Tension in supporting ring} = \frac{7.850}{5} \times 0.911 \times 41.25 = 59{,}000 \text{ lb.}$$

$$\text{Moment due to deflection} = 7{,}850 \times 0.435 = 3{,}410 \text{ lb. in.}$$

$$\text{Maxm. Stress} \quad \frac{2{,}500}{1.25} = 2{,}000 \text{ lb. per sq. in.}$$

$$\text{Plus} \quad \frac{3{,}410}{1.25} = 2{,}740 \text{ lb. per sq. in.}$$

$$\text{Plus} \quad \frac{7{,}850}{1.74} = 4{,}520 \text{ lb. per sq. in.}$$

$$9{,}260 \text{ lb. per sq. in.}$$

Fig. 23.—Test loading prototype

Approximate Calculation of Local Stress due to a Concentrated Load

The properties of the $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. tube are :—

a = overall depth	= 2.5 in.
area	= 1.74 sq. in.
Z modulus of section	= 1.25 ins.^{3}.
R = radius of gyration	= 0.9 in.

Assuming that in addition to the general loading of 30 lb. per sq. ft. one bay of rib carries also an additional load due to men walking on the roof and that this may amount to a load of 500 lb. spread along 5 ft.

Before the loaded rib can spread this overload to the other ribs it must span 5 ft. as a continuous beam. The loading will tend to flatten the rib. Following the L.U.D. Method with a factor of 2 on the limit of proportionality, the effective value of the flattening cannot exceed the L.U.D. and will be partly offset by the present upward camber.

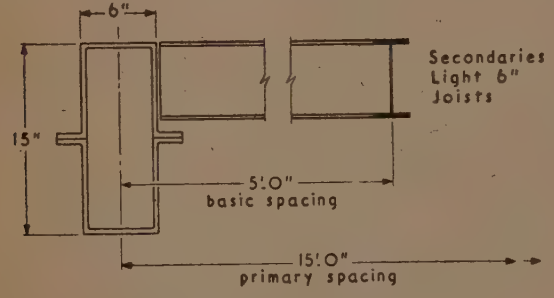


Fig. 24.—500 ft. dome

$$\text{L.U.D. (Downward)} = \frac{L^2}{1{,}790a} = \frac{60 \times 60}{1.90 \times 2.5} = 0.81 \text{ in.}$$

Camber (Upward) 0.375 in.

0.435 in.

Moment due to additional load

$$= 500 \times \frac{5}{12} \times 12 = 2{,}500 \text{ lb. in.}$$

Wrinkling Stress of a Thin Sphere

The point at which a long straight column becomes unstable under purely axial load was established by Euler many years ago. The point at which a uniform circular ring becomes unstable under uniform external pressure is also known but the point when a uniform spherical shell becomes unstable under uniform external pressure has apparently never been determined either by calculation or by experiment.

This factor alone made the construction of the first dome a hazardous operation.

Fig. 25.—Pitfodels dome-fixing sheeting

Tests on Full-scale Members of Prototype

A panel of sheeting 5 ft. $\times$ 5 ft. consisting of 16 gauge sheet of M57S$\frac{3}{4}$H alloy was fixed to a frame which

supported all four edges, and loaded with sand to 120 lb. per sq. ft. plus 400 lb. point load without any visible permanent distortion. As this loading was already five times the design load the test was carried no further.

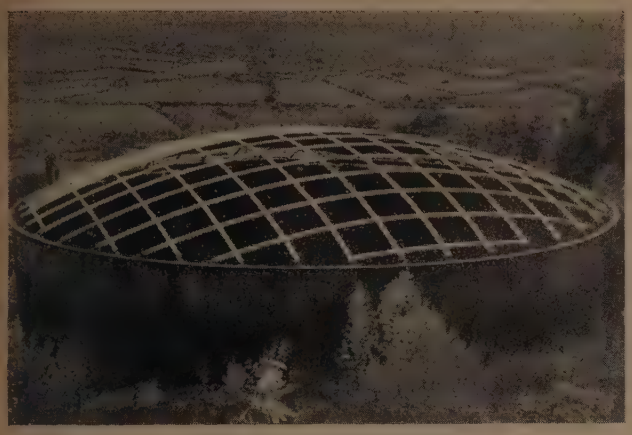


Fig. 26.—Tipperary dome

A later test on a larger sheet is shown in Fig. 12.

On April 15th, 1946, two compression tests were made on straight lengths of $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. tube with $2\frac{1}{4}$ in. diam. hollow.

The first on a plain length of tube 6 ft. long failed at 28.3 tons equal to 11.9 tons per sq. in. The second on a 6 ft. length of tube with a cross tube tenoned in at mid-length failed at 25.88 tons or 10.9 tons per sq. in. (Fig. 13).

On May 23rd, 1946, a further test on a 6 ft. plain length of $2\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in. tube with $2\frac{1}{8}$ in. $\times$ $2\frac{1}{8}$ in. hollow failed at 23 tons or 13.61 tons per sq. in.

In view of the fact that the calculated thrust was only $3\frac{1}{2}$ tons these tests were considered satisfactory.

Detailing of Prototype Roof

The calculation of the exact points of intersection is a rather long-winded application of spherical trigonometry. Starting with spacings of 5′—0″ on the two centre ribs a minimum of 4′—7.1″ is reached in the "corners" of the dome framing. The angle of intersection varies from 90° on the centre ribs to a maximum of 94° 52′. One of the working drawings is reproduced

Fig. 27.—Toronto dome

in Fig. 14. The exact location on the thrust ring of the bearing shoes at the ends of the ribs was calculated but this information was not required. Although the dimensions of the sides of the panels never exceed 5 ft. $\times$ 5 ft. the diagonal increases and exceeds $5\sqrt{2}$ ft., on lines at 45° to the main axes.

Fabrication of Prototype Roof

The curving of the main ribs was sub-let and proved expensive. For later roofs a home-made three-roller curving machine was "designed" and put together. Driven by a small motor, it curved the tubes quickly, accurately and cheaply.

The double tenon joints were cut by a double milling cutter mounted on a radial head and here a decision

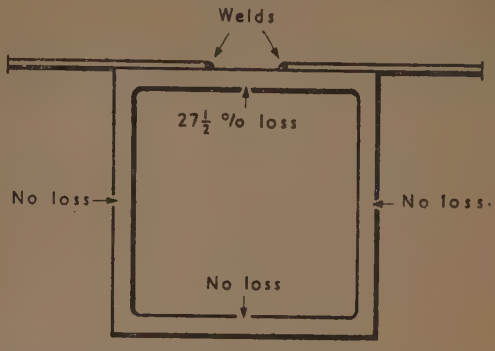


Fig. 28.—Reduction in strength due to welding heat

had to be made. There were variations of several thousandths in the overall width and thickness of the tubes and the sides tended to be slightly concave. Finally the standard cut shown in Fig. 15 was decided on.

Erection of Prototype Dome

Now that the strength of the Hamman dome has been so firmly established it is easy to discount one's early

Fig. 29.—Dingle Dome showing braced thrust ring

anxieties. Owing to delays beyond the authors' control, two years elapsed before the first dome was actually built. It was, as far as could be ascertained, the lightest structure in the world. Its method of construction was new and a complete break with traditional methods of constructing metallic framework. The material was only just coming into structural use and its ductility (10-13 per cent.) was less than that of mild steel. How close the structure was to the critical wrinkling stress

was (and still is) unknown. As it was essential that all the material used should be salvaged and re-used only temporary fastenings could be employed. A little comic relief was supplied by " experts " who prophesied :

(a) that the dome would burst and fly over the surrounding trees,

(b) that it would disintegrate due to internal temperature stresses.

Although the coefficient of expansion of aluminium alloys (about 0.000023 per 1°C.) is more than twice that

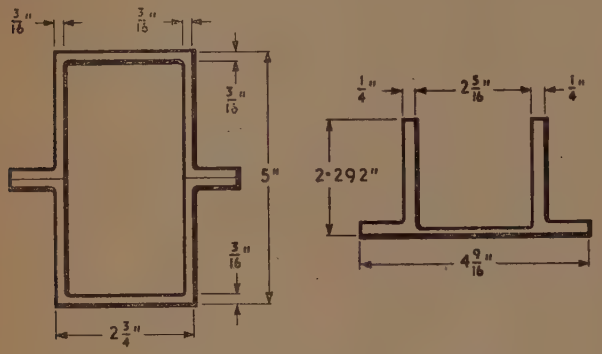


Fig. 30.—Main rib and (right) joint filler piece

of steel its elastic modulus is only one-third, so that although there is greater temperature movement there is less temperature stress than in a similar structure of mild steel.

The following difficulties were anticipated :—

1. The exact setting out of the bearing shoes on the thrust ring would be difficult.

2. The fact that the ribs were not curved exactly to 100 ft. radius would lead to serious distortion of the dome.

3. The fact that the ribs were in long lengths made it impossible for each joint to approach its mating joint radially as it must do if it is a push fit (see Fig. 16).

These three difficulties proved to be non-existent.

1. It is only necessary to set out the four shoes at the ends of the two centre ribs. As the remaining ribs are erected the shoes are brought up to their ends, the erection of the ribs automatically setting them out correctly.

2. If the lengths and angles are correctly set out and accurately cut the pushing home of the joints automatically produces an exact sphere.

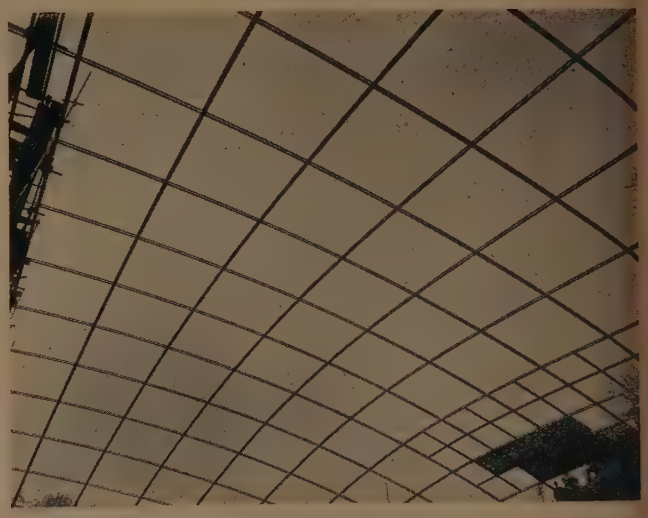


Fig. 32.—Eastbourne—Erection of secondaries and sheeting commencing

3. The ribs are sufficiently flexible to be twisted by hand to engage the mating joints.

Successive stages in the erection of the ribs and sheeting of the dome are shown in Figs. 17 to 22. The sheeting was fastened by small steel drive-screws. For permanent work both Chobert rivets and Linread " Rivnuts " were used. There are now on the market several types of " blind " rivet which may be successfully used.

When the erection of the ribs was complete and before sheeting commenced, attempts were made to apply a little pre-stressed compression to them by tightening up

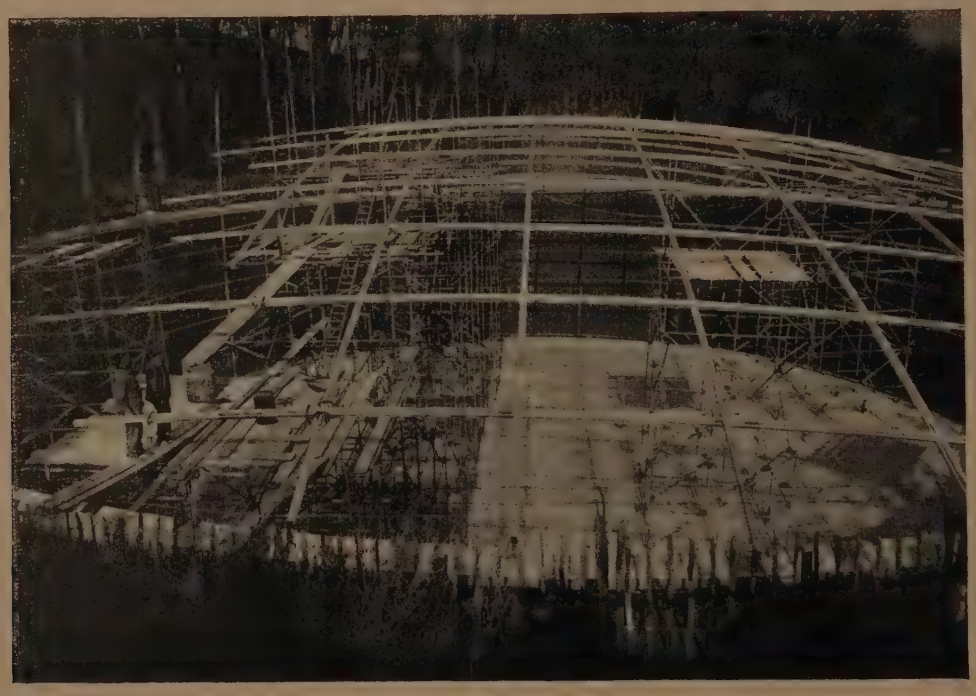


Fig. 31.—Eastbourne—Erection of main ribs

the end screws. This proved a complete failure. The crown of the dome merely bulged upwards and the tightening of any one screw merely lifted the ends of the two adjoining ribs out of their bearing sockets.

Testing of Prototype Dome

A loading of 30 lb. per sq. ft. had been rather vaguely specified and the dome was accordingly loaded with a layer of brick which, when wet, approximated to this figure. The loading was laid in rings commencing from the thrust ring and finishing at the crown (see Fig. 23). During the loading, deflections were measured at four points. Unfortunately, these were largely masked by temperature movement. The final deflection at the crown under full load was about 1 in., the maximum combined load and temperature movement being about 1.8 in.

Feeler-gauge measurements of the tenon joints after loading showed maximum gaps of the order 0.008 in.

Later, this dome was subjected to various arrangements of partial loading. Strain gauge readings indicated that the stress nowhere exceeded 9.333 lb. per sq. in. under any arrangement of loading.

Deductions from the Construction and Test-loading of Prototype Dome

After careful consideration of all the processes involved in the fabrication and erection of this roof and after further calculations, the following were established :—

1. Domes on the Hamman principle may certainly be built up to 500 ft. diameter. (See Fig. 24.)
2. They may probably be built to 600 ft. diameter.
3. They may possibly be built to 800 ft. diameter.
4. The distribution of stress in a latticed dome is extremely susceptible to small variations in the fitting of the joints between the members and between the members and the thrust ring. Small tolerances in either of these items may mean a variation of — 100 per cent. to +200 per cent. in the calculated load carried by any one rib.

In the design of latticed domes these two points are the vital and overriding considerations. Unless and until they are fully understood and unless and until active measures are taken to deal with them all attempts at mathematical analysis of the stresses in the dome are meaningless and misleading. It is interesting to note that the authors' engineering instincts led them at the outset to deal with these two points although there was no very evident reason for so doing until after the main ribs of the first dome were erected.

The most immediate and important requirement in theory is some fairly accurate method of calculating the critical wrinkling stresses as it is on this point that the possibility of still further reducing the sections of the main ribs depends.

Fig. 33.—Eastbourne—Riveting "closers"

Pitfodels Dome

This is 80 ft. diameter on the pitch circle and similar in construction to the prototype dome at Sheerwater. The ribs for the roof were erected, the scaffolding removed and the reservoir refilled in seven days.

Fig. 25 shows the process of fixing the sheeting after the reservoir was again in commission.

Tipperary Dome

The construction is similar to that of the prototype, the roof (Fig. 26) covering an existing steel tank 55 ft. diameter.

Dome at Toronto

Erected for the 1950 Fair, this is 83 ft. diameter. It was supported on twelve aluminium alloy stanchions of fancy lattice design and is the first dome to be assembled at ground level and hoisted into position. (Fig. 27.) This roof was later dismantled and re-erected at Southend.

Dome at Dingle

This is the first application of a Hamman dome to an oil tank and several new problems arose. The tank is 78 ft. 9 in. diameter.

Firstly, the roof has to resist working pressures of ±8 in. of water with a test pressure of ±12 in. water. This meant that it had to function under tension.

Secondly, the thrust ring, normally under tension, had to function under compression and its stability as a large thin ring arose.

Thirdly, the sheeting had to be welded to obtain a gas-tight roof.

The tensile strength of the 16-gauge sheeting was ample to take the internal pressure. The minor problem of fastening the sheets down round the periphery was solved by using an acute-angle extrusion.

Fig. 34.—Eastbourne—Main ribs complete

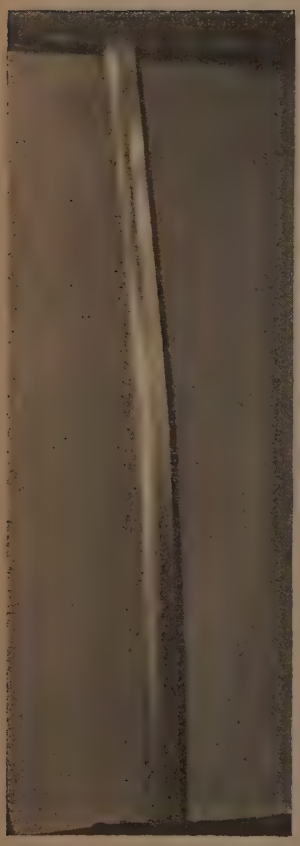


Fig. 35.—Eastbourne—Test on Plain's length of main rib

Fig. 36.—Eastbourne—Test on jointed length

The critical stability of a ring under uniform external pressure is

$$p = \frac{3EI}{R^3}$$

where p = unit external pressure
R = radius of ring

The thrust ring is made of two 4 in. × 2 in. steel channels laced together with 1 in. diameter rods welded on.

Fig. 37.—Lining plate

Earlier experiments with rivets had failed to give a gas-tight joint and some type of welding was essential. The Argon-arc process had proved successful under workshop conditions but this dome required welding over a large area in the open air and it was decided to use Murex electric welding. Tests having shown that

the welds were gas-tight the question of the reduction in strength of the main ribs had to be faced. Accordingly specimens were prepared and tested.

The local reduction in strength of the metal of the tube is shown in Fig. 28. A length of tube with " fins " of sheet welded on was tested as a strut and it was found that the strengthening effect of the addition of the

Fig. 38.—Roof over purifiers

" fins " was greater than the reduction in strength due to welding. Fig. 29 shows the main ribs and thrust ring.

Before the sheeting was finally fastened down round the edge the adjusting screws were tightened to give a certain amount of pre-stressed tension to the thrust ring.

Although primarily the tensile stresses under internal pressure of 8-in. of water are small and easy to calculate, there must be local variations due to deviations in shape of the tank and inequalities in workmanship. It was hoped to take a series of strain-gauge readings all round the edge of the tank to obtain some idea of what the maximum variation might be. Unfortunately, this could not be done and such strain-gauge readings as were taken were too local to be conclusive on this point.

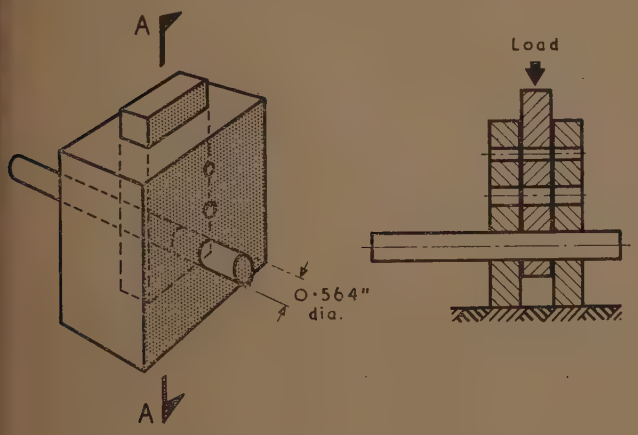


Fig. a.—Rig for shear test
Rod test pieces
Right—Section on A.A.

This roof is believed to be the largest all-welded aluminium structure in the world.

Small Domes in London

Two small domes 20 ft. and 30 ft. respectively in diameter were constructed over steel tanks, the main members being 1½ in. × 1½ in. Despite the fact that the main rib spacings were reduced to 3 ft. 9 in. and 4 ft. 3 in. respectively, the sheeting, owing to the strong curvature, was difficult to fix. When a third roof 25 ft. diameter was required it was found simpler to use a standard 2½ in. × 2½ in. tube curved to 75 ft. radius, the dome being similar in construction to the centre part of the dome in Fig. 26.

Dome at Eastbourne

Built over an existing reservoir, this roof is 151 ft. 10½ in. diameter measured on the pitch circle. The thrust ring is of reinforced concrete. As mild steel rods

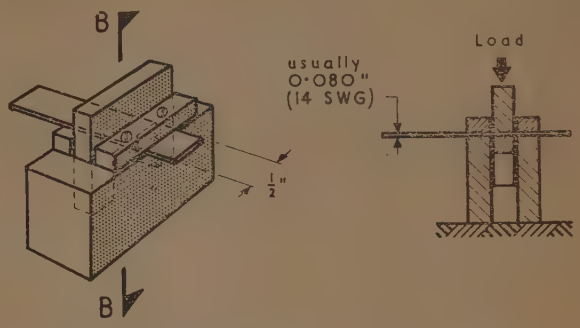


Fig. b.—Rig for shear test. Sheet test pieces
Right—Section on B.B.

were not available, small diameter wire rope was used for reinforcement.

It was realised at the outset that hollow extrusions were only suitable for comparatively small spans and the main ribs in this roof are formed by riveting two " top-hat " sections together (Fig. 30). As the two halves may be curved separately this presented no problem.

The main ribs are spaced basically at 10 ft. centres with secondary members dividing the surface into 5 ft. squares. Figs. 31 to 34 show details of the construction.

Before commencing fabrication two test pieces were assembled, one a plain length of rib 10 ft. long which crippled at a load of 44 tons, the other a jointed member which failed by crippling at 42 tons (Figs. 35 and 36). On pushing this second test beyond the failure point the rivets below the joint sheared.

The roof was erected by first placing the lower part of the north-south ribs in long lengths. The upper part of the east-west ribs were then placed over them in long lengths. The top parts of the north-south ribs and lower part of the east-west ribs were then cut to exact

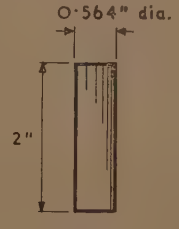


Fig. c.—Compression test piece for compression rig see Fig. d

length and placed in short pieces between the long lengths already placed. Special filler pieces are added at the joints so that, in effect, each rib runs full section through each joint (Fig. 30). Variations in the top-hat section, which tended to be wider at the " brim " than at the " crown," caused a small creep which had to be continuously adjusted and the method entailed a lot of site drilling and riveting but gave an interlocking joint. The sheeting was fastened by small Chobert rivets.

It is clear that the wind effect on these domes is in general very small although the authors do not know

of any trustworthy quantitative figures. The dome at Eastbourne, due to the special configuration of the surrounding terrain, may be an exception. Situated in a hollow scooped out under the brow of Beachy Head there may well be considerable discontinuity in the air-stream lines above the dome with appreciable uplift.

Fig. (d).—Compression specimen under test

The total roof weighs only 23 tons or slightly less than 3 lb. per sq. ft., so the thrust ring was anchored down with long bolts let into the existing walls.

Lined Roofs

The Hamman arrangement of members makes it simple to fix a lining plate to their underside (Fig. 37). Alternatively, plywood or similar lining board could be held by fixing strips attached to the underside of the ribs. The space between the inner and outer skin may be filled with insulating material such as " Solomite."

Relative Cost

General cost figures are almost always misleading, if not actually biased. In roofing work steel may be still slightly cheaper in first cost, although there is a steady tendency to close the gap. Adding the cost of normal painting over the effective life of the average roof brings the cost level. In situations where there is an acid atmosphere, or where the painting entails shutting down important plant, aluminium is much cheaper than steel.

Deterioration of Aluminium Alloys

None of the structures which the authors have designed has stood long enough to show any signs of deterioration either from the elements or from any anodic action (real or imaginary). The structure in Fig. 38 has stood for more than five years over a row of purifier boxes. After 2½ years' service the aluminium sheeting showed slight pitting to a maximum depth of 4.7 mils.

Much has been written on this subject and many experiments have been performed but the only really satisfactory way for any engineer to gain information is to build an aluminium structure as soon as possible and watch it carefully. Both the authors have aluminium structures in use.

Fire Hazards

In all situations where aluminium is likely to be employed any considerable fire will cause 100 per cent. loss. It is, therefore, neither better nor worse than any reasonable alternative form of construction. Cases where a fire is just sufficiently hot to cause loss of strength without serious distortion in aluminium framework could scarcely occur in practice. The fact that aluminium, when struck, cannot be made to spark is, in some circumstances, a point in its favour.

APPENDIX A

Shear, Bearing and Compression Tests

Figs. (a) and (b) show the shear blocks used for testing rod and sheet specimens. From these double shear tests single shear values are obtained.

The dimensions of the test specimen used for obtaining the proof stress in compression are shown in Fig. (c). A specimen is shown mounted in a sub-press between the platens of a testing machine in Fig. (d). The compressive strain is measured by the Martens mirror type instrument attached to the specimen.

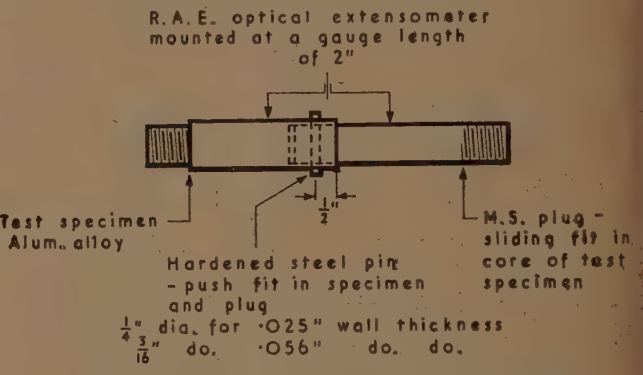

R.A.E. optical extensometer mounted at a gauge length of 2"
½"
Test specimen Alum. alloy
M.S. plug - sliding fit in core of test specimen
Hardened steel pin - push fit in specimen and plug
¼" dia. for ·025" wall thickness
3/16" do. ·056" do. do.

Fig. e.—Bearing test piece—Alum. alloy bar

A test piece used for bearing tests is shown in Fig. (e).

Acknowledgements

The authors are indebted to the following for permission to reproduce illustrations and information contained in this paper.

Aluminium Construction, Ltd.
The Aluminium Development Association.
Northern Aluminium Co., Ltd.
The City Engineer, Corporation of the City of Aberdeen.
The Chief Engineer, Tate & Lyle, Ltd.
The Chief Engineer, Regent Oil Co., Ltd.
Messrs. Binnie Deacon & Gourley.
The Ministry of Works.
The Chief Engineer, Esso Petroleum Co., Ltd.
The Divisional Engineer, Kent Division.
South-Eastern Gas Board.
R. H. Harry Stanger, A.M.I.C.E.
The British Oxygen Co., Ltd.
Arthur Guinness, Son & Co., Ltd.

Notes of Interest

Members of all grades are invited to submit "practice notes" of individual operations such as those described below, whose size and scope are not perhaps sufficient to justify a full-length article or paper, but which possess features of special or general interest to the Institution

Medium Span Portal Frame Reinforced Concrete Bridges in Singapore

By T. Karmakar, M.I.Struct.E.

General Description

The City of Singapore (City status conferred by Royal Charter on September 22nd, 1951) is famous as an entreport for the export of rubber and tin. Besides this, Communist territoristic activities on the mainland have focused the attention of the world to this island in recent years. The island is of hilly terrain and has a number of streams running through the city ; consequently many bridges of various types and spans are necessary to interlace the different parts for communication and transport. The construction of some of these bridges is different from usual practice, and this description may be of interest.

The grades of spans assumed in these notes are as follows :—Spans from 10 ft. to 30 ft. considered as short span, 30 ft. to 50 ft. as medium span and above 50 ft. taken as long span bridges.

The use of reinforced concrete portal frame type highway bridges of short span is quite common, but spans above 30 ft. clear are rare. The increased dead load of this type of bridge longer than 30 ft. span does not allow comparatively much reduction of bending moments at midspan. To overcome this disadvantage, a large amount of eccentricity in the abutment slab is introduced, which considerably increases the negative bending moments in the corner between deck slab and abutment, thus reducing the positive bending moments at midspan.

Other advantages gained with this arrangement are as follows :—The inward bend of the abutment slabs in some cases conforms with the profile of the banks of the stream, which considerably reduces the amount of excavation. The lower bending moments at midspan permit a reduced thickness for the deck slab, hitherto not possible with other types of construction, thereby decreasing the gradient at the approaches without sacrificing the maximum depth of headroom between water-level and the soffit of the deck. Due to the absence of ribs in most cases formwork was simplified, and last but not least, its pleasing æsthetic effect was a factor for its adoption.

On the above basis a few bridges were constructed by the then Municipality of Singapore (now City Council) between 1934 and 1939, and these are described in the following paragraphs.

Foundations

It is essential to have an unyielding rigid foundation to react against the considerable horizontal thrusts developed at the points between superstructure and foundations. The strip footings under the abutment slabs were connected by reinforced concrete ties at least three in number with large splays at the junctions of the footings to achieve additional rigidity. The probing of the ground by test borings was essential before determining the type of foundations required. Due to

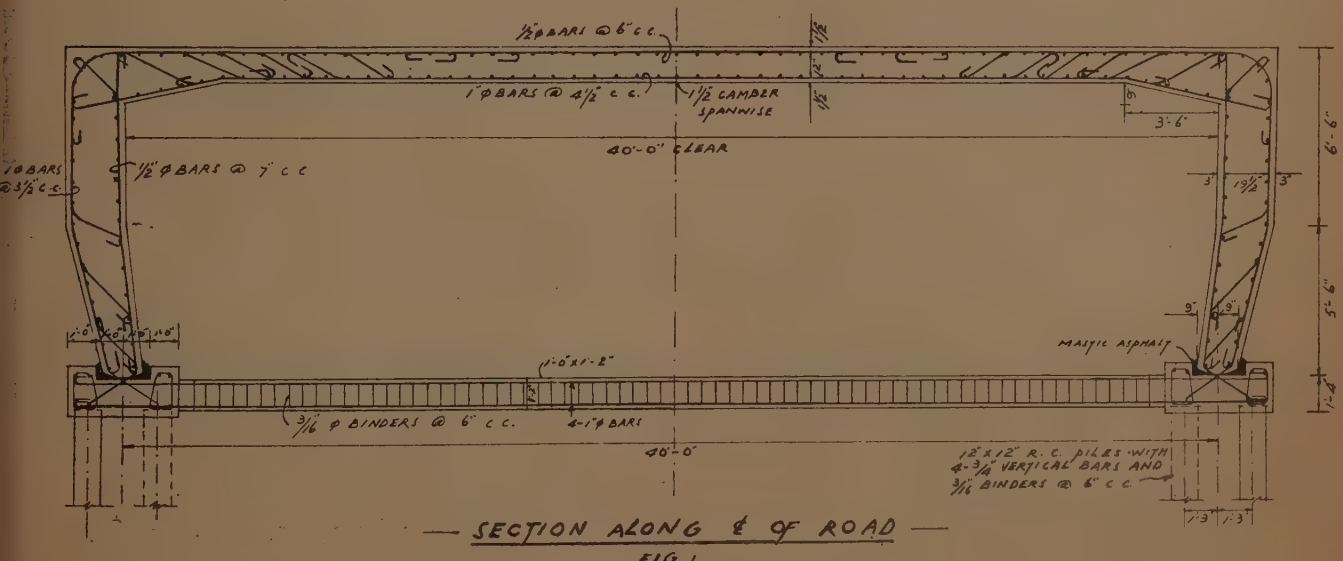
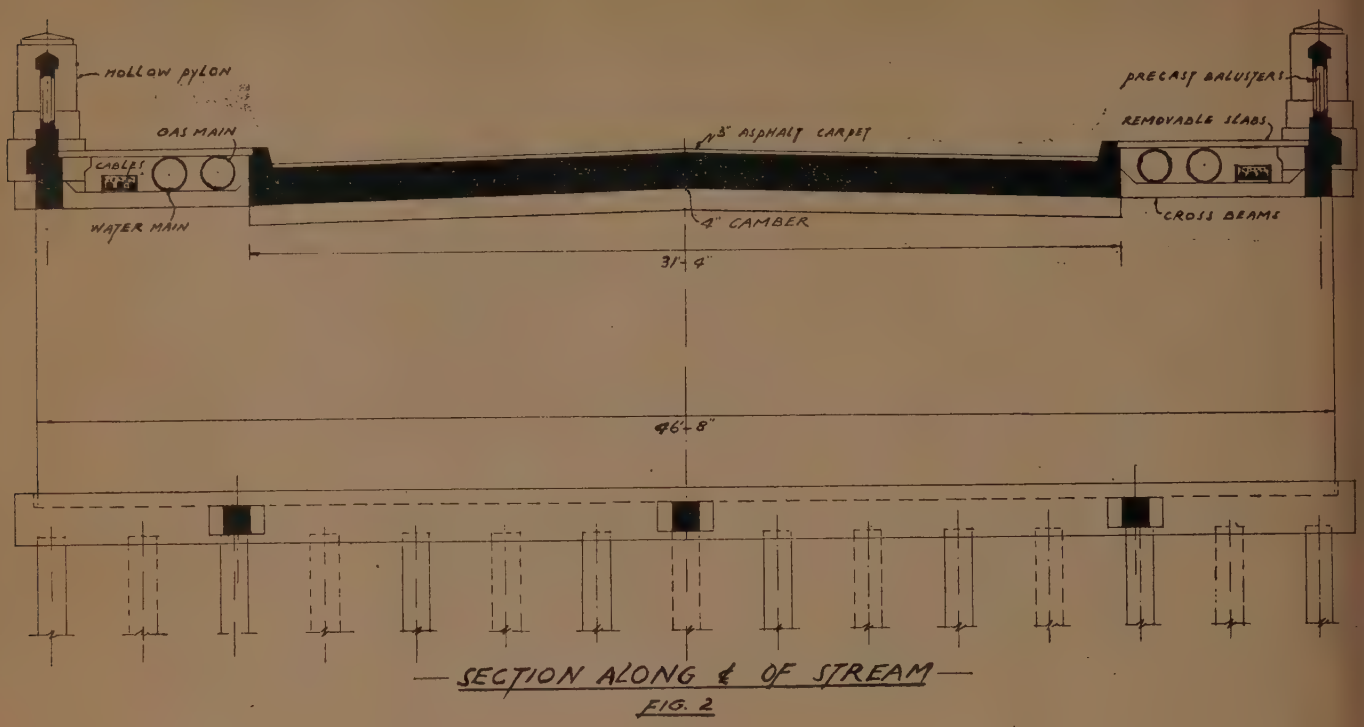

1/2" ϕ BARS @ 6" c.c.
1" ϕ BARS @ 4 1/2" c.c.
1 1/2" CAMBER SPANWISE
40'-0" CLEAR
1/2" ϕ BARS @ 7" c.c.
3/16 ϕ BINDERS @ 6" c.c.
4-1" ϕ BARS
40'-0"
MASTIC ASPHALT
12" x 12" R.C. PILES WITH 4-3/4" VERTICAL BARS AND 3/16 BINDERS @ 6" c.c.
— SECTION ALONG ℄ OF ROAD —

FIG. 1

the poor bearing capability of the soil under review, precast reinforced concrete piles were used. In every case the abutment slabs were hinged above the footings. The channel on the footings on which the abutment slabs stood was filled with hot mastic asphalt. (See Fig. 1.)



— SECTION ALONG ℄ OF STREAM —

FIG. 2

Design Stresses

The Ministry of Transport standard highway loading train was used for the live load. The allowable working stress for steel was 18,000 lb. per sq. in., and for concrete 1,000 lb. per sq. in. Steel bars in shear at hinges were taken as 5 tons per sq. in., neglecting resistance between the concrete surfaces.

Procedure of Construction

At Singapore a light shower is almost an everyday occurrence. The rainy season occurs twice a year, between June and August and between November and March with slight variations. The intensity of the latter is greater than the former. During this period the area under review often gets flooded, at times more than 36 hours at a stretch. Therefore the dry season of the year was invariably chosen for constructional purposes. During the period of construction, traffic was diverted to the next bridge, which in most cases was within half-a-mile from the proposed site.

A cofferdam with $1\frac{1}{2}$ in. thick plank sheet piling enclosing three sides of the foundation with provision for quick opening at the ends, in case of flood, was constructed before excavation for foundations commenced. After driving the reinforced concrete piles (which in some cases extended to a depth of 100 feet), a rectangular wooden chute resting on wooden trestles was provided to carry the dry weather flow of the stream. Dowels were left at the footings for hinges and horizontal ties; and in some cases the horizontal ties were cast along with the footings. Before the chute was removed the formwork for the vertical abutment slabs was constructed, providing 3 in. dia. weep holes in the abutments at the low tide level.

Fig. 3.—Arab Street Bridge
Clear Span 60 ft.
Ribs and slab portal frame R.C. bridge

Fig. 4.—Newton Circus Bridge
Clear span 40 ft.
All slab portal frame R.C. bridge

The formwork of the deck slab was carried out with a construction camber of 1½ in., which proved to be sufficient under full loads for a 40 ft. span.

To retain earth on the sides of the approaches, wings were constructed monolithically with the abutments in some cases. The back of the abutments was filled with puddle clay up to the level of the weep holes ; then dry packed with granite rubbles 12 in. to 18 in. thick carried up to the foundation of the road.

A 3 in.-thick asphalt carpet was rolled on the carriageway as a wearing surface. For footpaths, 3 in.-thick moveable reinforced concrete slabs were cast *in situ* with paper partitions to prevent adhesion to the concrete surface. Cross beams between the fascia and deck slab carried water and gas mains and electric cables under footpaths. (See Fig. 2.)

Precast balusters were placed in the groove on the fascia with steel projecting crosswise on the top which, with additional longitudinal bars, was concreted to form the top rail of the parapet. Hollow massive pylons were provided in four corners for æsthetic effect. The parapets, pylons and fascias were finished by rubbing briquets made of granite dust and cement in proportion of 1 : 1, with cement grout as lubricant, thereby producing a plaster-like finished surface.

The mix of concrete used was 1 : 2 : 4, with 3 in. to 4 in. slump. Crushed granite graded between 1½ in. to ¼ in., and clean sharp coarse sand was measured by box, and the cement was measured by weight. Pneumatic vibrators and ¾ in. dia. tamping rods were used during concreting. Out of every six batches of concrete four 4 in. cubes were made for testing, which were cured under field conditions and crushed on the 28th day. The ultimate crushing strength reached in most cases was 4,000 lb. per sq. in.

Photographs

Fig. 3 shows the Arab Street Bridge with a clear span of 60 ft., situated in one of the busiest parts of the city. Fig. 4 shows the Newton Circus Bridge on the main arterial route with a clear 40 ft. span.

Maintenance

Most of the bridges constructed have successfully stood the rough usage of military vehicles, including heavy tanks, during the last war. The only parts that suffered damage were the footpaths. These, of course, were easily replaced during the rehabilitation.

The bridges of this type have so far needed little or no repair, except periodical scrubbing of the parapets, pylons and fascias. The treatment is similar to the granite dust-cement briquets rub described above.

Acknowledgement

The author wishes to express his thanks to the City Engineer, Mr. G. Edmund, M.I.C.E., for permission to make use of the working drawings and photographs in respect of these notes.

Correspondence

The Institution, whilst being at all times pleased to open its columns to correspondence, cannot accept responsibility for the opinions expressed.

To the Editor of THE STRUCTURAL ENGINEER

The Analysis of Multi-Bay Gabled Rigid Frames By Arthur Bolton, April, 1953

Sir—Although I respect the technical virtuosity of Mr. Bolton's paper, I am not in sympathy with its intention which, as I understand it, is to present an alternative method of designing particular types of statically indeterminate frameworks.

It may be argued that there are already too many methods in use. I do not wish to pursue this argument. I am making the simpler proposition that now there are no advantages to be gained, either in accuracy, speed or understanding by departing from the principles of the classical method of evaluating the redundants along the lines proposed by Muller-Breslau, Ostenfeld and others.

I admit this has not always been so. In the past, hampered by illogical notation and laborious methods for solving the elasticity equations, engineers were not inclined to design frameworks in this way when there were more than three unknowns.

To-day, by using a more rational notation, it has become apparent the elasticity equations can be operated upon with ease and elegance by using matrix algebra. This algebra has long been used as a very powerful tool by applied mathematicians and physicists in their work but it is only in recent years that structural engineers have realised its value. R. S. Jenkins in 1947[1] showed that matrices not only simplify the analysis of the elasticity equations, but what is of the utmost consequence, they reveal the fundamental properties of the structural system.

It has meant that consideration of statically indeterminate frameworks can proceed symbolically up to the computation stage, thus eliminating the laborious preliminaries of, what I would term, the arithmetical methods.

As a simple illustration of this method, which has aptly been entitled the influence coefficient method, let us consider the frame shown in Fig. 1 (a). The redundants in this case can be evaluated from the following simultaneous equations :

$$\begin{aligned} U_1 + X_1 g_{11} + X_2 g_{21} + X_3 g_{31} &= 0 \\ U_2 + X_1 g_{12} + X_2 g_{22} + X_3 g_{32} &= 0 \\ U_3 + X_1 g_{13} + X_2 g_{23} + X_3 g_{33} &= 0 \end{aligned} \quad . \; . \quad (1)$$

where $g_{ik} = \int \frac{m_i m_k}{EI} ds$ and $U_i = \int \frac{M_p m_i}{EI} ds$

In matrix form equation (1) can be written :

$$\begin{bmatrix} g_{11} & g_{12} & g_{13} \\ g_{21} & g_{22} & g_{23} \\ g_{31} & g_{32} & g_{33} \end{bmatrix} \quad \begin{bmatrix} X_1 \\ X_2 \\ X_3 \end{bmatrix} = - \begin{bmatrix} U_1 \\ U_2 \\ U_3 \end{bmatrix}$$

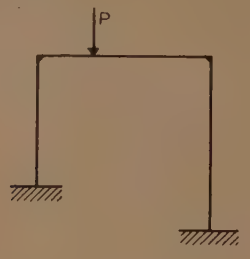
or $$GX = -U$$

It will be noted that side sway is dealt with directly and does not require separate consideration as is usual in the arithmetical methods.

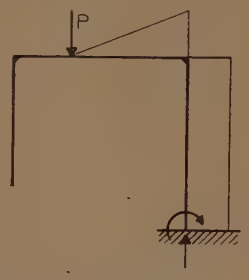
All that is now left is to find the unknowns by solving the elasticity equations :

$$GX = -U$$

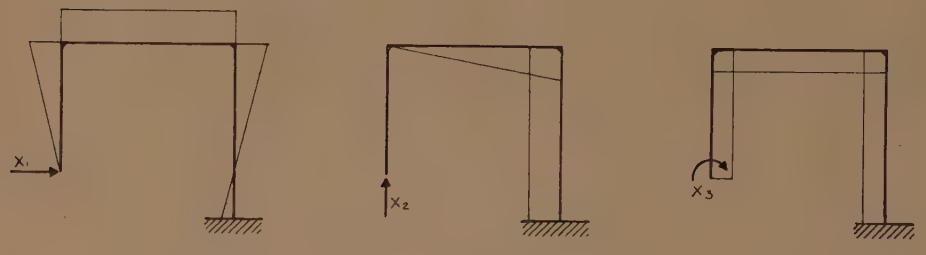
Now this equation, which is a positive definite symmetrical matrix, can be solved in a number of ways. Fox in (2) has summarised this position mainly for mathematicians but in (1) is given a detail arrangement for computation. This method is extremely rapid and I can recommend it to designing engineers who usually have little time to spare.



(a) THE FRAME



(b) THE STATICALLY DETERMINATE SYSTEM



(c) MOMENTS DUE TO THE REDUNDANTS

To sum up, I contend the influence coefficient method has universal validity in the solution of statically indeterminate frameworks. It is a natural method which allows the problem to be visualised in the simplest possible terms. It clearly reveals the applied force-displacement relationship of the members comprising the framework. It is therefore to be preferred to arithmetical methods, which tend to obscure, if not obliterate, the manner in which the structure works.

References

[1]R. S. Jenkins. "Theory and Design of Cylindrical Shell Structures." 1947.

[2]L. Fox. "Practical Methods for the solution of linear simultaneous equations and the inversion of matrices." London, Royal Statistical Society. Journal of the Society (Series B). Vol. 12. 1950.

Yours, etc.,
PETER DUNICAN
(Associate Member).

Economy in the Use of Structural Materials

By G. A. Gardner, O.B.E., M.I.Struct.E., R. Morton, B.Eng., M.I.C.E. and L. R. Creasy, B.Sc., M.I.C.E.

In the report of the discussion on the above paper, which was published in the September issue of the Journal, the remarks made by Mr. C. V. Blumfield (Member) were wrongly attributed to Mr. E. H. Bloomfield (Associate-Member).

Mr. Blumfield's contribution to the discussion should read as follows :—

"Mr. C. V. BLUMFIELD (Member), after commenting on the great interest of the subject and the valuable information that had been given, endorsed very heartily some of the points raised in the paper.

"First he commented on the general point concerning the length of time allowed for the preparation of the design and details. He felt that one of the most important functions of the Institution of Structural Engineers should be to endeavour to get clients or architects into a frame of mind whereby they would give the unfortunate engineer time to prepare details before the job started instead of having to do that during the job, as so frequently happened.

"It was very interesting to observe the introduction of welded tubular steelwork into modern structural methods, and he felt there was a tremendous future in that type of construction. It had many advantages, a major one of them being its flexibility in construction ; by that he meant that it could be welded and constructed with comparatively small plant, without the tremendous set-up of the usual steelwork fabricating shop. Apparently Mr. Bedford made the various parts of his structures on the site as the erection proceeded, which was a very interesting experiment, and the length of time required for fabrication appeared to be very much less than for conventional steelwork.

"One was very interested to hear Mr. Bedford speak of the beam/column connection. Mr. Blumfield said he had been responsible for the design and construction of a building of rather the same nature in which the connection of the beam to the column was arranged to come on saddles, so that any slight discrepancy in the length of the beam could be taken up. Connections were made with couplers of the type used in scaffolding, so that there was no difficulty in matching bolt holes and the erection was carried out very easily."

Institution Notices and Proceedings

ORDINARY GENERAL MEETING

An Ordinary General Meeting of the Institution of Structural Engineers was held at 11, Upper Belgrave Street, London, S.W.1, on Thursday, October 22nd, 1953, at 5.55 p.m., Lt.-Colonel R. F. Galbraith, M.C., B.Sc.(Eng.), M.I.Struct.E., A.M.I.C.E. (President), in the Chair.

The Minutes of the Ordinary General Meetings held on May 28th and June 25th, 1953, as published in the Journal, were taken as read, were confirmed and signed.

The following members were elected in accordance with the Bye-Laws. Will members kindly note that the elections as tabulated below should be referred to when consulting the Year Book for evidence of membership?

GRADUATES

BISS, Russell George, of Bristol.
BROMIGE, Keith Bernard, of Ruislip, Middlesex.
BURTON, Thomas Edmund, of Bridlington, Yorks.
CHORLEY, Victor Wilfred, of Bexley, Kent.
COVERMAN, Sidney Harold, of London.
DUNN, Horace William James, of London.
JABLONSKI, Piotr, of London.
MYERS, Leslie Walter, of London.
PRINGLE, Alan, of Sunderland, Co. Durham.
SMITH, Raymond Charles Henry, of London.
TODD, Paul, of Birmingham.
WAGNER, Arthur Hiram, of London.

ASSOCIATE-MEMBERS

COPPEN, Frank, of London.
DICKINSON, John George, A.M.I.C.E., of Newcastle upon Tyne.
FARMER, Lawrence Theodore, of Beckenham, Kent.
HAQUE, Mohammed Manzurul, B.E.(Civil) Calcutta, of Derby.
HOLT, Hermann Paul, of London.
MACINTYRE, John Ross McDermit, of Holytown, Lanarkshire.
MORRELL, Donald Beaumont, of London.
OVERTON, Kenneth Albert, of West Bromwich, S. Staffs.
RAE, Richard George, A.M.I.C.E., A.M.I.Mun.E., of Grenada, Windward Islands.
REDLICH, Walter, B.Sc.(Eng.), London, D.I.C., of Hemel Hempstead, Herts.
RILEY, Thomas William, of Plymstock, Devon.
ROBINSON, Ernest Arthur Feakes, of Enfield, Middlesex.
ROSS, Colin Forrester, B.E.M., B.Sc.(Civil) Birmingham, of London.
SOMERSET, Francis Eyre, of South Ruislip, Middlesex.
SPOORS, Colin, of West Hartlepool, Co. Durham.
WEBSTER, Stanley John, of Bromley, Kent.

TRANSFERS

Students to Graduates

BARTLEY, Raymond, of Watford, Herts.
BRADLEY, Dennis, of Wembley, Middlesex.
CARNAN, Lewis Rippon, of Consett, Co. Durham.
CORBETT, Joseph Frank, of Bury, Lancs.
EGWUATU, Gordon Chukwuka, of London.
HOOKHAM, Raymond Albert, of Eastbourne, Sussex.
IDOWU, Charles Olatunji, of London.
KAY, Terence, of Chapeltown, near Sheffield.
KIRBY, Norman, of Gateshead, Co. Durham.
MORGAN-GREEN, Peter, of London.
NASH, Alan Blake, of Caterham, Surrey.
NEWBY, Ronald William, of London.
REFFITT, Michael John Joseph, of Harrogate, Yorkshire.
TARATULA, Ryszard Leopold, of London.
UNDERWOOD, Bernard George, of Berkhamsted, Herts.
WILKS, Leslie Robert, of London.
WINFIELD, Peter Frederick, of Osterley, Middlesex.
WOOLLARD, Malcolm William Thomas, of Buckhurst Hill, Essex.

Graduates to Associate-Members

ADASKO, Stanislaw, of Manchester, Lancs.
ANDERSON, Jack Ernest, of Pinner, Middlesex.
BINDER, Paul Frank, of Skewen, Glamorgan.
BOWERING, Brian Arthur, of Liverpool.
BROWN, Cecil Allen, of London.
BROWN, David Henry, of Hazel Grove, nr. Stockport, Cheshire.
BULLOCK, Leslie, A.M.I.C.E., of London.
COBB, Walter Clifford, of Middlesbrough, Yorks.
CORDING, John Stanley, of Normanby, nr. Middlesbrough, Yorks.
DRUMMOND, Richard, of Glasgow.
EVANS, Maurice Hambling, B.Sc.(Hons.) Birmingham, of Birmingham.
GAUNT, Grenville, of Chesterfield, Derbyshire.
HALL, Thomas Robert, B.Sc. Belfast, of Dublin.
HANSEN, John, of Epsom, Surrey.
HINDER, Stanley James, of Bristol.
HOLMES, Ian Miles, of Twickenham, Middlesex.
HUGGINS, Raymond Arthur, B.Sc.(Eng.), London, of London.
KIRKMAN, Cyril John, of New Barnet, Hertfordshire.
LEACH, Alan Brickett, of Watford, Herts.
LOADSMAN, Roy Vernon Charles, of London.
LONGMAN, Donald James, B.Sc.(Eng.) London, of Beckenham, Kent.
MAYO, George Edward, of Merstham, Surrey.
MORRIS, David Austin, B.Sc.(Eng.) London, of Rustington, Sussex.
MURPHY, William, B.E., N.U.I., of Seven Kings, Essex.
PREECE, Stanley George, of Woodford Green, Essex.
RAGGETT, Leonard John, of Hemel Hempstead, Herts.
SAMPSON, Derek Stanley, of Denham, Middlesex.
SAYERS, Ronald Ernest Courtney, of Bexleyheath, Kent.
SCHOLNICK, Gabriel, B.Sc. Cape Town, of London.
SILHAN, Stanley George, of Culcheth, Lancs.
SIMPSON, Morrison, B.Sc.(Eng.) London, of Wilford, Notts.
SLEE, John Penry, B.Sc.(Eng.) London, of London.
SMART, James Desmond, of London.
SMEDLEY, Granville Barrie, B.Sc.(Tech.) Manchester, of Blackpool, Lancs.
TATE, Alan George, B.Sc.(Eng.) London, of Beckenham, Kent.
WALDEN, Kenneth Owen, B.Sc.(Eng.) London, of London.
WALKER, Charles Henry, of Runcorn, Cheshire.
WATTIE, Neville, of Salford, Lancs.
WILLCOX, Leslie George, of North Walsham, Norfolk.

FORTHCOMING MEETINGS

The following forthcoming meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Thursday, December 17th, 1953

Ordinary General Meeting for the election of members, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m.,

when Mr. W. Hamilton, A.M.I.C.E., and Mr. G. P. Manning, M.Eng., M.I.C.E., will give a paper on "Some Structural Uses of Aluminium Alloy with special reference to Domes."

POSTPONEMENT OF MEETING

Owing to unavoidable circumstances, it has been necessary to postpone the Ordinary Meeting which was to have been held on the 14th January, 1954.

Thursday, January 28th, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. E. J. Callard, M.A., M.I.Mech.E., will give a paper on "Design and Construction of a Welded Portal Frame Warehouse Building Designed by the Plastic Method."

Thursday, February 25th, 1954

Ordinary General Meeting, 5.55 p.m., followed by an Ordinary Meeting at 6 p.m., when Mr. R. A. Sefton Jenkins, B.Sc., A.M.I.C.E., A.M.I.Struct.E., A.C.G.I., will give a paper on "Prestressed Steel Lattice Girders."

Members wishing to bring guests to the Ordinary Meetings announced above are requested to apply to the Secretary for tickets of admission.

EXAMINATIONS, JULY, 1953

Overseas Centres

The examinations were held overseas in July, 1953, at the following centres :—

Accra, Auckland, Beirut, Bombay, Bulawayo, Cairo, Calcutta, Cape Town, Chittagong, Christchurch (N.Z.), Dar-es-Salaam, Dunedin, Durban, Germany (B.A.O.R.), Hong Kong, Johannesburg, Kampala (Uganda), Kingston (Jamaica), Kulala Lumpur, Lahore, Madras, Mombasa, Nairobi, Nicosia (Cyprus), Port Elizabeth, Salisbury (S. Rhodesia), Singapore, Toronto, Wellington (N.Z.).

Twenty-four candidates took the Graduateship Examination, and 86 took the Associate-Membership Examination, a total of 110. Of these, 15 passed the Graduate ship Examination, and 19 passed the Associate-Membership Examination. The names of the successful candidates are :—

Graduateship Examination

Cheng Hon Kwan.
Deshpande, Vasant Balkrishna.
Harris, Colin Frederick.
Herring, Bernard Carroll.
Keller, George Charles Chapman.
Mills, Gordon Ernest.
Nassar, Joseph Jiries.
Pugh, Trevor George.
Richardson, Paul Herbert.
Rutherford, Michael William.
Singh, S. Balbir.
Snethlage, Frederik.
Thistle, Sydney Alan.
Tonner, Eric William.
Yeo Lim Hui.

Associate-Membership Examination

Balachandran, Nelliat Koroth.
Balsara, Rustom Jehangir.
Chandramouli, Tunuguntla.
Chokhavatia, Ratilal Harilal.
Cragg, Wilfred Shipham.
Fairman, Charles Edward John.
Halford, Frederick Samuel.
Hudson, Hilbert Leslie.
Karanjia, Minoo Rustamji.
Louw, Timotheus.
Mahfouz, Georges Elias.
Morley, Kenneth Outram.
Morrow, James Robert.
Morten, Peter.
Nichols, John Russell.
Pinto, Vitorino Antonio C. L.
Rathod, Mahendra Valjibhai.
Walker, Eric Stuart.
Yue Shun-Kwong.

EXAMINATIONS—JANUARY, 1954

The Examinations of the Institution will next be held at centres in the United Kingdom and Overseas on January 5th and 6th (Graduateship), and Januray 7th, and 8th, 1954 (Associate-Membership).

PERSONAL

A member of the Institution wishes to obtain, on behalf of a Library, copies of past issues of the Journal prior to 1927. If any member has such copies for disposal it would be appreciated if he would kindly communicate with the Secretary.

MACLACHLAN LECTURE COMPETITION

The closing date for the receipt of entries for the MacLachlan Lecture Competition is Wednesday, March 31st, 1954. The general conditions of the competition are as follows :

1. The Institution of Structural Engineers shall institute a written lecture to be known as the MacLachlan Lecture, and to be held annually.

2. The subject of the Lecture may be on any aspect of Structural Engineering as long as in every second year the subject shall be confined to steel structures.

3. Entrance into the Competition for the Lecture shall be confined to Associate Members of the Institution, who are under the age of 32 years.

4. All papers entered for the competition shall be submitted to assessors to be appointed by the Council of the Institution, and all such papers (including the prize-winning Lecture) shall be available for publication in the Journal of the Institution at the discretion of the Council.

5. No paper submitted shall have been published or read elsewhere.

6. The winner of the competition shall be required to present the Lecture to a meeting of the Institution at which he will be presented with the sum of £17 10s.

7. Should a competitor's paper be considered worthy of ranking second in merit he will receive a consolation award of £5.

8. In the event of there being no winner of the competition in any one or more years, whether because no lecture submitted is considered to be of sufficient merit to warrant award, or for any other reason, the Institution shall transfer the above sums to the Research Fund of the Institution.

Particulars of the Competition for 1954

1. The MacLachlan Lecture will be given at a meeting of the Institution to be arranged towards the end of 1954.

2. The subject of the Lecture shall be on any aspect of structural engineering.

3. The work should be submitted as the script of a lecture which the author, if successful in the competition,

will deliver before an audience in the course of about one hour. The development of mathematical formulæ and detailed calculations should be avoided as far as possible in the text ; if they are essential they should be embodied in appendices. Photographs, drawings, graphs, etc., which would appear as illustrations to the lecture in published form, should accompany the script. If additional illustrations would be shown as slides, a list of these should be included.

4. Six copies of each Lecture should be submitted and should be addressed to the Secretary of the Institution.

5. The closing date for the receipt of entries by the Institution is Wednesday, March 31st, 1954.

EXAMINATIONS

Preparation for the Examinations of the Institution by Attendance at Technical Colleges

A candidate for Graduateship or Associate-Membership may be able to attend a technical college ; these notes are intended to guide him in choosing the most suitable instruction.

Preparation for the Graduateship Examination

Technical Colleges offer :

(a) Full-time courses for degrees of Higher National Diplomas in Building or Engineering.

(b) Part-time day or evening courses for Higher National Certificates in Building or Engineering.

If he obtains a Higher National Certificate or Diploma complying with Appendix II, Section V, of the Regulations Governing Admission to Membership, the candidate will be exempted from the Graduateship Examination.

Alternatively, he may study subjects selected from the available courses and sit the Graduateship Examination. At technical colleges courses are usually available in Building Science or Engineering Science, Strength of Materials, Theory of Structures and Surveying, but students are not normally allowed to select subjects from National Diploma or Certificate courses unless they can show evidence of sound training in more elementary studies. The advice of the College Authorities should be followed.

Preparation for the Associate-Membership Examination

At some technical colleges there are part-time courses in Structural Engineering which cover the syllabus of the Associate-Membership Examination. At other colleges the candidate must rely on Higher National Certificate courses or on advanced courses in Building, Civil Engineering or Municipal Engineering ; these cover only part of the requirements for the Associate-Membership Examination.

Colleges in List " A " provide at least two years of instruction in Theory or Structures and in Structural Engineering Design and Drawing up to Associate-Membership standard. They also give instruction in Structural Specifications, Quantities and Estimates.

List " A "

Belfast College of Technology.
Birmingham College of Technology.
Bolton Municipal Technical College.
Bradford Technical College.
Bridgend Technical College.
Chesterfield College of Technology.
Derby Technical College.
Dudley and Staffordshire Technical College.
Glasgow Royal Technical College.
City of Liverpool College of Building.
L.C.C. Brixton School of Building, S.W.4.
L.C.C. Hammersmith School of Building and Arts and Crafts, W.12.
Manchester College of Technology.
Middlesbrough, Constantine Technical College.
Salford, Royal Technical College.
South-West Essex Technical College, Walthamstow, E.17.
Stockport College for Further Education.
Twickenham Technical College.
Willesden Technical College, N.W.10.

Colleges in List " B " provide instruction in Theory of Structures from which the student may reach Associate-Membership standard, but instruction in Structural Engineering Design and Drawing and in Structural Specifications, Quantities and Estimates is not usually so complete.

List " B "

Brighton Technical College.
Cardiff Technical College.
Edinburgh, Heriot-Watt College.
Huddersfield Technical College.
Leeds College of Technology.
London, Battersea Polytechnic, S.W.11.
London, Northampton Polytechnic, E.C.1.
L.C.C. Westminster Technical College, S.W.1.
Plymouth and Devonport Technical College.
Preston, Herris Institute.
Wigan Mining and Technical College.
Woolwich Polytechnic, S.E.18.

Students are advised to take the organised courses in Structural Engineering where these are available.

LONDON GRADUATES' AND STUDENTS' SECTION

The following meetings will be held at 11, Upper Belgrave Street, London, S.W.1.

Tuesday, December 8th, 1953

Mr. S. Turner, on " The Foundation to an 8,000,000 cu. ft. Gas-holder." 6 p.m.

Tuesday, January 12th, 1954 (*Provisional*)

Mr. E. G. Robinson, A.M.I.Struct.E., on " The Design of Chimneys."

Tuesday, February 16th, 1954

Address by the President of the Institution.

Hon. Secretary : J. F. S. Pryke, B.A.(Hons.), Bushcroft, Slipe Lane, Wormley, Herts.

BRANCH NOTICES

Lancashire and Cheshire Branch

The following meetings have been arranged :—

Tuesday, January 12th, 1954

Joint Meeting with the Institute of Welding, Liverpool and District Branch, when a paper on " The Plastic Theory and its Application to Welded Structures," will be given by a member of the staff of the British Constructional Steelwork Association, at the Liverpool College of Technology, at 7.15 p.m.

Wednesday, January 27th, 1954

Joint Meeting with the Institution of Civil Engineers, North-Western Association. Mr. Edgar Morton, M.Sc., on " Site Exploration and Foundation Problems."

Thursday, February 11th, 1954

Dr. F. G. Thomas, B.Sc., M.I.C.E., M.I.Struct.E., on " Composite Action in Structures."

All meetings, unless otherwise stated, will be held in the Reynolds Hall, College of Technology, Manchester, at 6.30 p.m., preceded by tea at 5.45 p.m.

Joint Hon. Secretaries: A. S. Sinclair, A.M.I.Struct.E., 17, The Circuit, Cheadle Hulme, Cheshire.

M. D. Woods, A.M.I.Struct.E., 58, Spring Gardens, Salford, 6.

MIDLAND COUNTIES BRANCH

The following meetings have been arranged :—

Friday, January 22nd, 1954

Mr. W. G. Carter, M.B.E., B.Sc., A.M.I.C.E., A.M.I.Struct.E., on "True Economies in Planning, Designing and Detailing of Structures."

Tuesday, February 9th, 1954

Mr. E. Shepley, B.Sc., A.M.I.C.E., on "Prestressed Concrete Framework for Liverpool University Medical School," at the Supper Room, The King's Hall, Queen Street, Derby, 7.0 p.m.

Friday, February 26th, 1954

Mr. C. B. Brewington, B.Sc., A.M.I.C.E. (Graduate), and Mr. J. W. Fortey, A.C.T. (Birmingham), A.M.I. Struct.E., on "A Method of Structural Analysis by Large-Scale Models."

All meetings, except where otherwise stated, will be held in the James Watt Memorial Institute, Great Charles Street, Birmingham, at 6.0 p.m.

Hon. Secretary : L. A. Firminger, A.M.I.Struct.E., 656, Chester Road, Erdington, Birmingham, 23.

GRADUATES' AND STUDENTS' SECTION

The following meetings have been arranged :—

Friday, January 29th, 1954

Mr. D. H. New, B.Sc., M.I.C.E., A.M.I.Mech.E. (Member of Council), on "Some Practical Applications of Prestressed Concrete," at the James Watt Memorial Institute, Great Charles Street, Birmingham, at 7 p.m.

Hon. Secretary : H. L. Bramwell, 139, Wood Lane, Handsworth Wood, Birmingham, 20.

NORTHERN COUNTIES BRANCH

The following meetings have been arranged :—

Tuesday, December 1st, 1953

At Middlesbrough, Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., (Member of Council), and Mr. K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E., on "Reconstruction of a Soaking Pit Building."

Wednesday, December 2nd, 1953

The above meeting will be repeated at Newcastle.

Wednesday, January 6th, 1954

Joint Meeting with the Northern Architectural Association (at Newcastle).

Wednesday, January 13th, 1954

Joint Meeting with the Institution of Civil Engineers. Mr. A. P. Clark on "Lackenby Works."

Tuesday, February 2nd, 1954

Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., on "Prestressed Concrete in Civil Engineering Works" (at Middlesbrough).

Wednesday, February 3rd, 1954

The above meeting will be repeated at Newcastle.

All Meetings will commence at 6.30 p.m., the Middlesbrough Meetings being held at the Cleveland Scientific and Technical Institution, Corporation Road, and the Newcastle Meetings in the Neville Hall, near the Central Station.

Hon. Secretary : O. Lithgow, A.M.I.Struct.E., 4, Stoneleigh Avenue, Acklam, Middlesbrough.

NORTHERN IRELAND BRANCH

The following meetings have been arranged :—

Tuesday, January 12th, 1954

Details to be announced.

Tuesday, February 9th, 1954

Annual Dinner and Social Function, at the Grand Central Hotel, Belfast, 6.0 p.m. Visit of the President and the Secretary of the Institution.

Hon. Secretary : A. H. K. Roberts, B.A., B.A.I., M.I.Struct.E., M.I.C.E.I., "Barbizon," 26, Dunlambert Park, Belfast.

SCOTTISH BRANCH

The following meetings have been arranged :—

Thursday, December 3rd, 1953

At Elite Hotel, Windsor Street, Edinburgh, at 6 p.m. Mr. James Richey, M.C., B.A., Sc.D., on "Alkali Reaction in Concrete."

Tuesday, December 15th, 1953

At the Ca'doro Restaurant, Glasgow, 6 p.m. Details will be circulated to members.

Monday, January 18th, 1954

At the Ca'doro Restaurant, Glasgow, 6.30 p.m. Details to be announced. Visit of the President and the Secretary of the Institution.

Tuesday, January 19th, 1954

Annual Dinner and Dance at Grosvenor Restaurant, Gordon Street, Glasgow. The President and the Secretary of the Institution will be present.

Tuesday, February 9th, 1954

At the Ca'doro Restaurant, Glasgow, 6 p.m. Details to be announced.

Thursday, February 25th, 1954

At the Elite Hotel, Edinburgh, 6 p.m. Mr. Hugh B. Sutherland, S.M.(Harvard), A.M.I.C.E., F.G.S., A.M.I.Struct.E., on "Some Problems in Foundation Engineering."

Hon. Secretary : G. Drysdale, A.M.I.Struct. E., 33, Union Street, Motherwell, Lanarkshire.

SOUTH-WESTERN COUNTIES BRANCH

The following meetings have been arranged :—

Friday, December 11th, 1953

At Exeter, 7 p.m. Mr. H. J. Scoles, L.R.I.B.A., M.I.Struct.E., on "Gap Graded Concrete."

Friday, January 15th, 1954

At Newton Abbot, 7 p.m. Mr. J. C. Woodrow (Graduate, Joint Hon. Secretary), on "The Tamar Bridge Proposal."

Friday, February 19th, 1954

At the Duke of Cornwall Hotel, Plymouth. 7 p.m. Mr. A. V. R. Hooker, A.M.I.C.E., M.I.Struct.E., on "Structural Engineering at Abbey Works, Margam."

Joint Hon. Secretaries : E. W. Howells, M.I.Struct.E., 10-12, Market Street, Torquay, Devon.

C. J. Woodrow, "Elstow," Hartley Park Villas, Tavistock Road, Plymouth, Devon.

WALES AND MONMOUTHSHIRE BRANCH

The following meetings have been arranged :—

Tuesday, December 8th, 1953

At Swansea. Meeting followed by films.

Tuesday, January 26th, 1954

At Cardiff. Structural Quiz.

Wednesday, January 27th, 1954

At Swansea. Structural Quiz.

Wednesday, February 17th, 1954

At Swansea. Junior Members' Evening.

Meetings at Cardiff will be held at the South Wales Institute of Engineers, Park Place, at 6.30 p.m.

Meetings at Swansea will be held at the Mackworth Hotel at 6.30 p.m.

Hon. Secretary : G. R. Brueton, A.M.I.C.E., A.M.I.Struct.E., 86, The Exchange, Mount Stuart Square, Cardiff.

WESTERN COUNTIES BRANCH

At the opening meeting of the Session, held at the University of Bristol, on Friday, October 2nd, members had the pleasure of meeting the President of the Institution, Lt.-Colonel R. F. Galbraith, M.C., and the Secretary, Major R. F. Maitland, O.B.E. The Chairman, Mr. E. N. Underwood, B.Sc., M.I.C.E., M.I.Struct.E., gave the address on "Further Problems in Practice." A vote of thanks to the Chairman was proposed by Mr. N. G. T. Ball, M.I.Struct.E., (Vice-Chairman).

After the meeting, the Chairman, together with members of the Branch, entertained the President and the Secretary at an informal dinner at the Royal Hotel, Bristol.

The following meetings have been arranged :—

Friday, December 4th, 1953

Combined Dance at the Royal Hotel, Bristol.

Friday, January 8th, 1954

Mr. A. J. Harris, B.Sc.(Eng.), A.M.I.C.E., on "Hangars at London Airport—Design of Large-span Prestressed Concrete Beams."

Friday, February 5th, 1954

Mr. F. E. Collins, A.R.I.B.A., A.M.I.Struct.E., on "The Industrial Architect and Engineer."

Wednesday, February 17th, 1954

Annual Dinner, at the Royal Hotel, Bristol.

Unless otherwise stated, all meetings will be held in the University of Bristol Geology Lecture Theatre, at 6 p.m., preceded by tea at 5.30 p.m.

Hon. Secretary : E. Hughes, A.M.I.Struct.E., 23, Southdown Road, Westbury-on-Trym, Bristol, 9.

YORKSHIRE BRANCH

The following meetings have been arranged :—

Wednesday, December 16th, 1953

Mr. W. N. Espie, A.M.I.Struct.E., on "The Handling of Bulk Cement."

Wednesday, January 20th, 1954

Mr. H. C. Husband, B.Eng., M.I.C.E., M.I.Mech.E., (Member of Council), and Mr. K. H. Best, B.Eng., A.M.I.C.E., A.M.I.Struct.E., on "The Reconstruction of a Soaking Pit Building."

Wednesday, February 17th, 1954

Mr. H. D. Morgan, M.Sc., M.I.C.E., on "Driving and Testing of Piles."

The above meetings will be held at the Great Northern Hotel, Leeds, at 6.30 p.m.

Hon. Secretary : E. Wrigley, A.M.I.Struct.E., 17, The Drive, Alwoodley, Leeds.

UNION OF SOUTH AFRICA BRANCH

Branch Hon. Secretary : A. E. Tait, B.Sc., A.M.I.C.E., A.M.I.Struct.E., P.O. Box 3306, Johannesburg. During week-days Mr. Tait can be contacted in the City Engineer's Department, Town Hall, Johannesburg. 'Phone: 34-1111, Ext. 257.

Natal Section Hon. Secretary : E. G. Bennett, A.M.I.Struct.E., c/o The Reinforcing Steel Co., Ltd., P.O. Box 49, Merebank, Durban.

Cape Section, Hon. Secretary : R. Stubbs, M.I.Struct.E. The Reinforcing Steel Co., Ltd., P.O. Box 2692, Cape Town.

ADDITIONS TO THE LIBRARY

Acknowledgement is hereby made of the generous gift by Sir F. Arthur Whitaker, K.C.B., of the following volumes from the library of his brother, the late Mr. H. A. Whitaker, Past Chairman of the Lancashire and Cheshire Branch and of the Yorkshire Branch.

AITKEN, A. C. *Determination of Matrices*, 5th Ed., 1948.

ALLCOCK AND JONES. *The Nomogram*, 4th Ed., 1950.

AMERIKIAN, A. *Analysis of Rigid Frames*, 1942.

BELDEN, E. S. *Building Superintendence for Steel Structures*, 1917.

BLEICH, F. R. *Theorie und Berechnung der Eisernen Brucken*, 1924.

BLEICH, F. R. *Die Berechnung statisch unbestimmter Tragwerke nach der Methode des Viermomentensatzes*, 1918.

BOYLE, R. *Geology in Relation to Civil Engineering* (Glasgow Technical College, 1910).

BUREAU OF STANDARDS. *Results of some Compression Tests of Structural Steel Angles* (Technologic Paper No. 218).

CASSIE, W. Fisher. *Structural Analysis : The Solution of Statically Indeterminate Structures*, 1947.

CHARNOCK, G. F. *Graphic Statics*, Vols. I, II and III.

COURANT, R. *Differential and Integral Calculus*, 2nd Ed., 1940.

CROSS and MORGAN. *Continuous Frames of Reinforced Concrete*, 1932.

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH. *Steel Structures Research Committee Reports*, Nos. I, II and III.

DUNHAM, C. W. *Theory and Practice of Reinforced Concrete*, 2nd Edition, 1944.

FABER, Oscar. *Examples of Steel Design under the new Code of Practice*, 1934.

FOERSTER, Max. *Repetitorium fur den Hochbau*, Vols. I, II and III.

GRANHOLM, H. *On Composite Beams and Columns with particular reference to Nailed Timber Structures* (In Swedish, with Summary in English), Chalmers Univ. Trans., 1949.

GREGOR, A. *Der Praktische Eisenhochbau*, Vols. I, II and III.

GRINTER, L. E. *Automatic Design of Continuous Frames in Steel and Reinforced Concrete*, 1939.

GRINTER, L. E. *Design of Modern Steel Structures*, 1941.

GRINTER, L. E. *Theory of Modern Steel Structures*, Vols. I and II, 1936-7.

Anon. *Handbuch fur Eisenbetonbau*, Vols. I, II, III and VI.

HAYDEN, A. G. *The Rigid Frame Bridge*, 1931.

HAWKEN, R. W. *Column Analysis and Design* (Univ. Queensland Proc., 1918).

HAWKEN, R. W. *Column Design Curves and Correspondence* (Inst. Eng. Australia, 1927).

HAWKEN, R. W. *Critical Planes and Curves of Rupture in Cohesive Material* (Inst. Eng. Australia, 1928).

HOOL and KINNE. *Movable and Long-span Bridges*, 1923.

HOOL and KINNE. *Stresses in Framed Structures*, 1923.

INCE, E. L. *Integration of Ordinary Differential Equations*, 1949.

ITERSON, F. K. Th. van. *Plasticity in Engineering*, 1947.

KLOUCEK, C. V. *Distribution of Deformation*, 1949.

KETCHUM, M. S. *Design of Highway Bridges*, 2nd Ed., 1920.

KETCHUM, M. S. *Design of Walls, Bins and Grain Elevators*, 1919.

KLEINLOGEL, A. *Bewegungsfugen im Beton-und Eisenbetonbau*, 2nd Ed., 1938.

KRISO, K. *Statik der Vierendeeltrager*, 1922.

LEGAT, DUNN and FAIRHURST. *Design and Construction of Reinforced Concrete Bridges*, 1948.

LUNDGREN, H. *Cylindrical Shells*, Vol. I—*Cylindrical Roofs*, 1949.

MANNING, G. P. *Reinforced Concrete Arch Design*, 1933.

MCCULLOUGH, C. B. *Economics of Highway Bridge Types*, 1929.

MERRIMAN, M. and WOODWARD, R. S. *Higher Mathematics : A Textbook for Classical and Engineering Colleges*, 1900.

Anon. *Niagara Falls Suspension Bridges, etc.* (John Weale, 1856.)

OTZEN, R. *Der Massivbau* (Stein-, Beton- und Eisenbetonbau), 1926.

PAASWELL, G. *Retaining Walls—Their Design and Construction*, 1920.

PIPPARD and BAKER. *Analysis of Engineering Structures*, 2nd Ed., 1943.

PARCEL and MANEY. *Elementary Treatise on Statically Indeterminate Stresses*, 2nd Ed., 1936.

RATHLING, G. *Tabellen zur Berechnung Durchlaufender Trager unabhangig von der Anzahl der Felder und dem Verhaltnis der Spannweiten*, 1931.

RANKINE, W. J. M. *A Manual of Civil Engineering*, 6th Ed., 1869.

RUTHERFORD, D. E. *Vector Methods applied to Differential Geometry, Mechanics and Potential Theory*, 7th Ed., 1951.

SALMON, E. H. *Materials and Structures*, Vol. 2—1938.

SEJOURNE, P. *Grandes Voutes*, Vols. I, II, III, IV, V and VI, 1913.

SOUTHWELL, R. V. *Relaxation Methods in Engineering Science*, 1943.

SOUTHWELL, R. V. *Relaxation Methods in Theoretical Physics*, 1946.

STAACK, J. *Rahmen und Balken*, 1931.

STEED, R. W. *An Introduction to Distribution Methods of Structural Analysis*, 1950.

TIMOSHENKO, S. *Theory of Elastic Stability*. 1936.

TIMOSHENKO, S. *Theory of Plates and Shells*, 1940.

TIMOSHENKO, S. *Vibration Problems in Engineering*, 2nd Ed., 1937.

TIMOSHENKO and YOUNG. *Theory of Structures*, 1945.

VAN DEN BROEK, J. A. *Elastic Energy Theory*, 1931.

WENTWORTH-SHEILDS and GRAY. *Reinforced Concrete Piling*, 1938.

WHITELAW, J., Jun. *Surveying as practised by Civil Engineers and Surveyors*, 3rd Ed., 1910.

Institution of Civil Engineers, Selected Engineering Papers, Nos. 8, 71, 80, 83, 99, 101, 106, 112, 118 and 119

"Der Bauingenieur," 1928, 1929, 1930, 1931 and 1932.

Book Review

Introductory Soil Mechanics and Foundations, by George B. Sowers and George F. Sowers. (New York : McMillan. 1951.) 284 pp. 9½ in. × 6 in. Price 22s.

Amongst the abundant literature on Soil Mechanics it is refreshing to find a book which is not the "mixture as before." The authors of this work have set themselves out to deal with the subject primarily from the practical standpoint. The first page recounts a simple little story illustrating the fact that Soil Mechanics can help the practical engineer to avoid costly mistakes. The remainder of the book describes how this can be done.

After dealing in the first three chapters with the nature, physical properties and geological aspects of soils, the authors proceed to describe the various engineering problems to which Soil Mechanics can be applied with advantage.

Site exploration and testing are described in sufficient detail for those who are not specialists, and who do not want to become involved in higher mathematics. It is rightly assumed that the practical engineer will leave such matters to the specialist.

The nature, properties and geology of soil are dealt with thoroughly and with such exceptional clarity that they are worth a study by all ; be they students, practical engineers or specialists. The same observations can be made about the chapters dealing with the application of Soil Mechanics. These not only describe how Soil Mechanics is applied but also provide the reader with most of the basic knowledge of civil engineering works which have any connection with the natural ground.

Particular mention should be made of Chapter 6, dealing with deep foundations and piling. The latter is a subject of which most writers on Soil Mechanics seem to have fought shy in the past. Admittedly, the design of piled foundations is still surrounded with a certain amount of uncertainty, but the authors have however made a real effort at tackling the problem in a scientific and practical manner.

The authors are to be congratulated not only on packing in an extraordinary amount of useful knowledge in a comparatively small book, but also upon presenting it in such a lucid and interesting manner. This book should appeal alike to the student, the specialist and the experienced engineer.

C. B. B.